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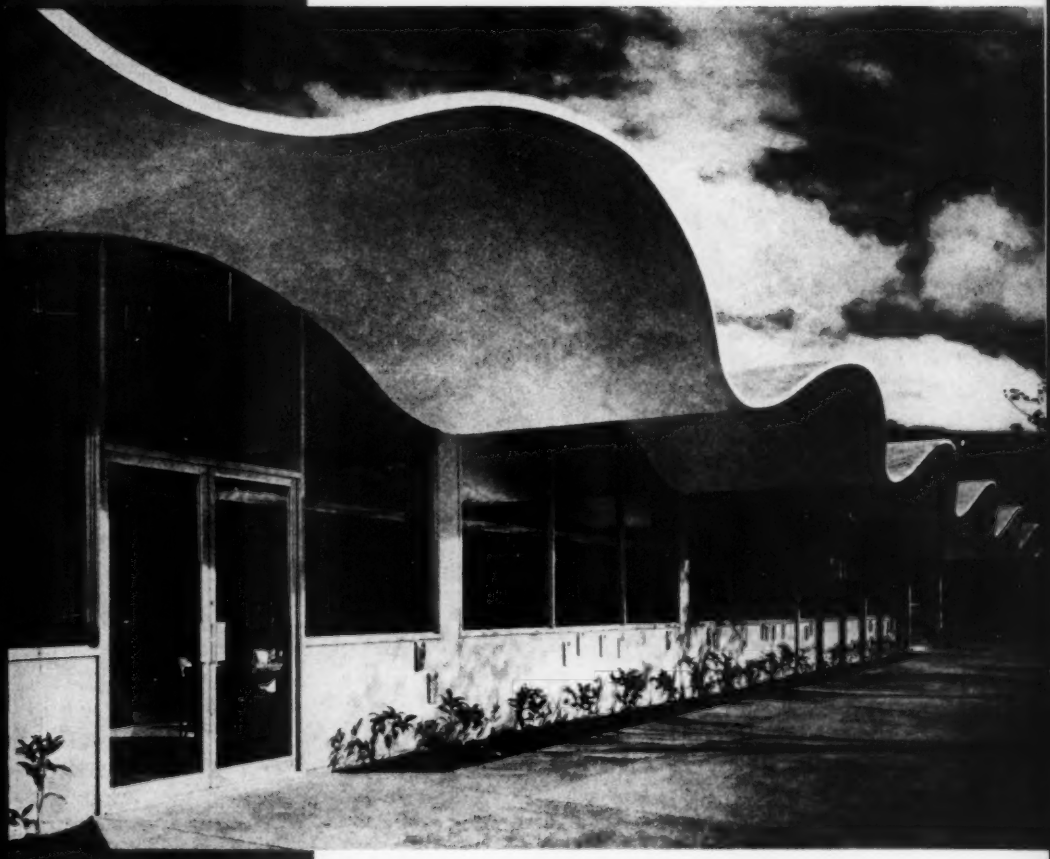

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On the Cover

Striking view of a wave form shell in Honolulu, Hawaii. The shells are 2 in. thick and the total axial span varies between 60 and 120 ft. A more complete description on wave form shells and shells in general is given starting on p. 129.

Coming next month in the JOURNAL

FRANK W. CHAPPEL highlights the design-construction story of the "Masonic Home and School Chapel" in Fort Worth, Tex., describing special foundation design considerations necessitated by highly expansive soil.

●

ELMER C. HIGGINSON reports on an intensive investigation conducted in the Bureau of Reclamation laboratories, Denver, on the "Effect of Steam Curing on the Important Properties of Concrete."

●

Design data for "Concrete Shear Walls Combined with Rigid Frames in Multistory Buildings Subject to Lateral Loads," are presented in a paper by BERNHARD CARDAN.

●

Results of research applying theoretical lateral buckling relationships to a prestressed concrete girder to determine the validity of the theory and the limitations of the assumptions are reported by WALTER PODOLNY, JR. and JOHN B. SCALZI in their paper, "Lateral Stability of a Prestressed Concrete Girder."

●

In "Thermal and Shrinkage Stresses in Composite Beams," by WILLIAM ZUK, equations are developed for both longitudinal and transverse stresses in composite beams under various conditions of temperature and shrinkage.

●

Discussion of papers published in the January, February, and March, 1961, issues will appear in the September JOURNAL in accordance with the Institute's quarterly publication schedule for *Proceedings* discussion.

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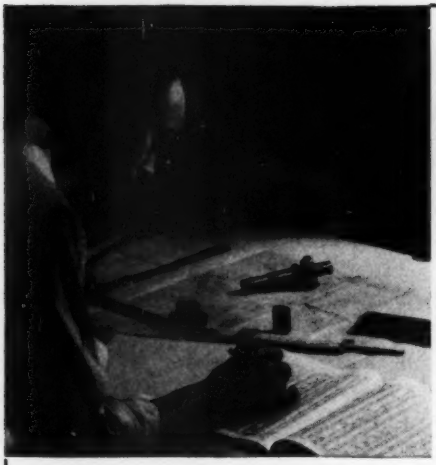

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Title No. 58-5

Application of the General Theory of Shells

By RICHARD R. BRADSHAW

General differential equations of a shell are presented with a method of solution for same. The bending and twisting moment terms are included. The boundary conditions are considered in the solution. The shell has no ribs or hinges. Also, a more rational approach to buckling of concrete shells is presented with consideration being given to large deflection theory of buckling and plasticity effects. The construction and behavior of the shell are discussed.

■ IN THE ANALYSIS of a doubly curved shell for a market near Honolulu, it became necessary to use the general differential equations of a shell. This was necessitated by the fact that the shell varied in thickness and because membrane theory would have led to stiffening ribs. Furthermore, it was believed that the bending stresses were of an order of magnitude of the membrane stresses. The solution to the equations showed this to be the case (see Table 3). While the equations for special types of shells and special cases of loading are readily available, such as cylindrical, hyperbolic paraboloid, shells of revolution, and membrane theory, the general equations for any shell including bending terms are not so well known; nor are the solutions to these equations readily available. Hence, it was necessary to derive and solve these equations. Since both the equations and their solution are general, the method employed could have wide application.

GENERAL EQUATIONS

Notation

u = deflection in the direction of the x-axis
v = deflection in the direction of the y-axis
w = deflection in the direction of the z-axis
ν_c, ν = Poisson's ratio
ν_p = Poisson's ratio, plastic, -0.5
t = thickness of shell
q = load per sq ft
R_1, R_2 = the principal radii of curvature
D = flexural rigidity of shell = $Et^3/12(1-\nu^2)$
H = increment length in the finite difference net*
h = H/R
σ_{xx} = stresses due to stretching of middle surface of shell in x-direction

*Note the terms grid, mesh, and net are used interchangeably in discussions of the relaxation procedure.

σ_{ye} = stresses due to stretching of middle surface of shell in y-direction
σ_{xb} = bending stresses in x-direction
σ_{yb} = bending stresses in y-direction
τ_{xye} = direct shear stresses
τ_{xyb} = twisting moment shears
N_{xe} = axial load on an element in the x-direction
N_{ye} = axial load on an element in the y-direction
M_{xb} = bending in x-direction on an element
M_{yb} = bending in y-direction on an element
M_{xyb} = twisting moment
Q_{xye} = tangential shear
a_o = initial imperfection of shell
d = diameter of shell (measured in plan dimension) participating in buckling
n = number of half buckles across a shell
U_o = an uneveness factor
σ_{CL} = critical buckling stress by classical equations
σ_{CR} = critical buckling stress by using experimental and theoretical studies of large deflection theory
$E = E_s$ = secant modulus of elasticity of concrete
E_t = tangent modulus of elasticity
η = plasticity effect in buckling

The general differential equations of a shell consist of three simultaneous, linear, partial differential equations, two of the second order and one of the fourth order. These equations may be derived by proceeding from equilibrium and compatability conditions in a manner similar to that by which the equations for special cases have been derived,[1,2] or they may be more easily derived by extending the equations for a cylindrical shell. This extension is made by making the assumption that cross products of the two radii of curvature which appear in the denominator are small in comparison with the other terms. It should be pointed out that those who have presented shell equations have given equations which differ from one another. The reason for this is that certain terms are always dropped out because they are of a small order of magnitude, or because the particular shell discussed could afford to drop these terms. In our case we will extend the equations derived by Donnell[1] to the general case of double curvature. Donnell's equations are:

$$\left.\begin{aligned}
&\frac{\partial^2 u}{\partial x^2} + \frac{(1-\nu)}{2}\frac{\partial^2 u}{\partial y^2} + \frac{(1+\nu)}{2}\frac{\partial v^2}{\partial x \partial y} - \frac{\nu}{R_1}\frac{\partial w}{\partial x} = 0 \\
&\frac{\partial^2 v}{\partial y^2} + \frac{(1-\nu)}{2}\frac{\partial^2 v}{\partial x^2} + \frac{(1+\nu)}{2}\frac{\partial^2 u}{\partial x \partial y} - \frac{1}{R_1}\frac{\partial w}{\partial x} = 0 \\
&\frac{t^2}{12}\nabla^4 w + \frac{1}{R_1}\left(\frac{w}{R_1} - \frac{\partial v}{\partial y} - \nu\frac{\partial u}{\partial x}\right) - \frac{t^2}{12D}q = 0
\end{aligned}\right\} \quad \text{.....(1)}$$

Since these differ somewhat from those of Timoshenko, on solution of Donnell's equations the answers were used to check the results of dropping out certain of Timoshenko's terms.* From this it was seen that these terms had only a small influence on the answer.

*Note that the nomenclature of Reference 2, Eq. (257), p. 440, uses polar coordinates for derivatives in the y-direction and a for R; hence $a\partial\phi = \partial y$.

ACI member **Richard R. Bradshaw** is president of Richard R. Bradshaw, Inc., structural engineers, Los Angeles. He has been responsible for the design of numerous structures all over the world including thin shells and other space frame works of unusual design. He is a member of ACI Committee 334, Concrete Shell Structures, and ACI-ASCE Committee 328, Limit Design.

Extending Eq. (1) to the case of double curvature we get

$$\left.\begin{aligned}
&\frac{\partial^2 u}{\partial x^2} + \frac{(1-\nu)}{2}\frac{\partial^2 u}{\partial y^2} + \frac{(1+\nu)}{2}\frac{\partial v^2}{\partial x \partial y} - \frac{\nu}{R_1}\frac{\partial w}{\partial x} - \frac{1}{R_2}\frac{\partial w}{\partial x} = 0 \\
&\frac{\partial^2 v}{\partial y^2} + \frac{(1-\nu)}{2}\frac{\partial^2 v}{\partial x^2} + \frac{(1+\nu)}{2}\frac{\partial^2 u}{\partial x \partial y} - \frac{1}{R_1}\frac{\partial w}{\partial y} - \frac{\nu}{R_2}\frac{\partial w}{\partial y} = 0 \\
&\frac{t^2}{12}\nabla^4 w + \frac{1}{R_1}\left(\frac{w}{R_1} - \frac{\partial v}{\partial y} - \nu\frac{\partial u}{\partial x}\right) + \frac{1}{R_2}\left(\frac{w}{R_2} - \frac{\partial v}{\partial x} + \nu\frac{\partial u}{\partial y}\right) - \frac{t^2 q}{12D} = 0
\end{aligned}\right\} \quad (2)$$

where $\nabla^4 = \partial^4 w/\partial x^4 + 2\partial^4 w/(\partial x^2 \partial y^2) + \partial^4 w/\partial y^4$ = the biharmonic expression for bending of a flat plate. We note that letting R_1 or R_2 go to infinity, we get the general equations of a cylindrical shell. By letting both R_1 and R_2 go to infinity, we get the general equations of a flat plate. By letting $R_1 = R_2$ we get the general equations of a spherical shell. By letting $R_1 = R_2 = \infty$ and the width equal unity, we get the general differential equation of a beam, i.e., $EI\, d^4w/dx^4 = q$. Hence, Eq. (2) represents general equations suitable for many structures.

In our case, the shell is composed of the intersection of two tori. Since the major radius of the tori is 240 ft and the minor radius is 120 ft, we have the intersection of two pinhole tori and $R_1 = R_2/Z$ (see Fig. 1). Also, we will now make a further assumption that can be shown to have only a small effect on the answer, i.e., let $\nu = 0$ for concrete.

This greatly simplifies our equations and we thus have:

$$\left.\begin{aligned}
&\frac{\partial^2 u}{\partial x^2} + \frac{1}{2}\frac{\partial^2 u}{\partial y^2} + \frac{1}{2}\frac{\partial v^2}{\partial x \partial y} - \frac{1}{2R_1}\frac{\partial w}{\partial x} = 0 \\
&\frac{\partial^2 v}{\partial y^2} + \frac{1}{2}\frac{\partial^2 v}{\partial x^2} + \frac{1}{2}\frac{\partial^2 u}{\partial x \partial y} - \frac{1}{R_1}\frac{\partial w}{\partial y} = 0 \\
&\frac{t^2}{12}\nabla^4 w + \frac{1}{R_1}\left(\frac{w}{R_1} - \frac{\partial v}{\partial y}\right) + \frac{1}{2R_1}\left(\frac{w}{2R_1} - \frac{\partial v}{\partial x}\right) - \frac{t^2}{12D}\, q = 0
\end{aligned}\right\} \quad (3)$$

For simplicity we shall now nondimensionalize these equations by using the following relationships:

$$\bar{x} = \frac{x}{R_1}\,,\quad \bar{y} = \frac{y}{R_1}\,,\quad \bar{u} = \frac{u}{R_1}\,,\quad \bar{v} = \frac{v}{R_1}\,,\quad \bar{w} = \frac{w}{R_1} \quad (4)$$

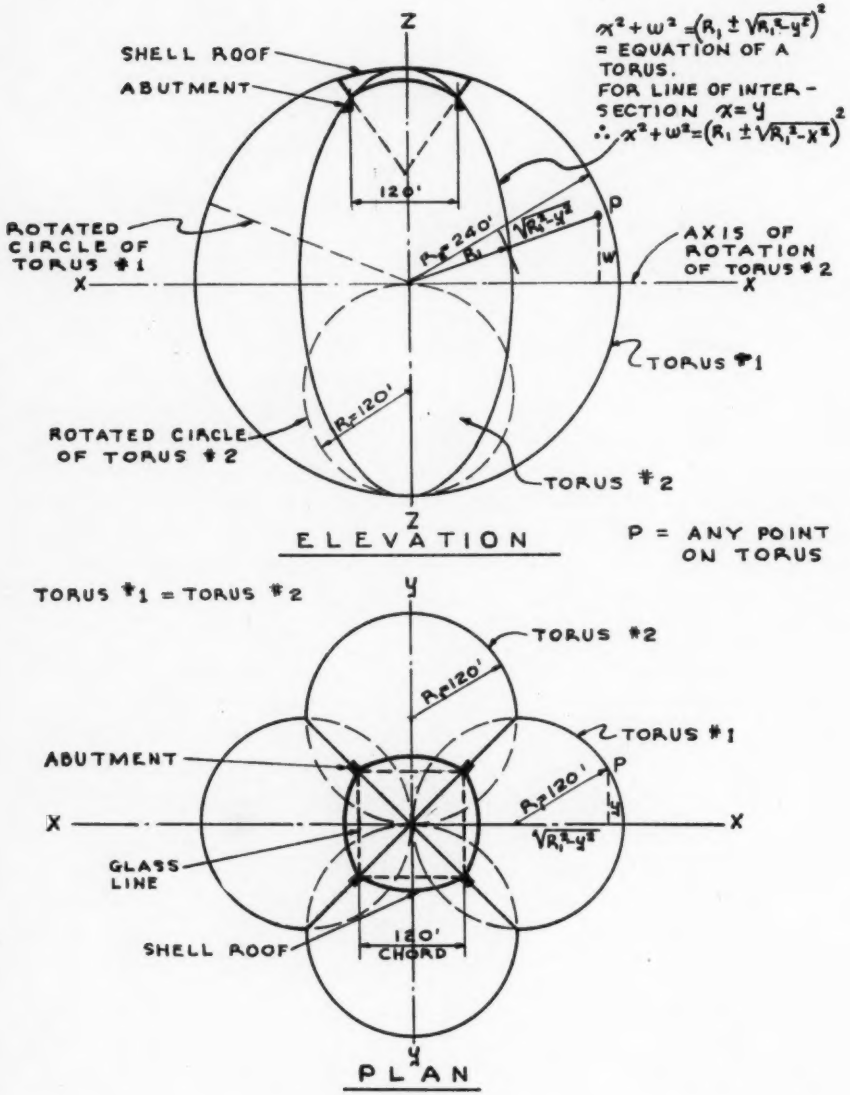


Fig. 1—Shell composed of intersection of two tori

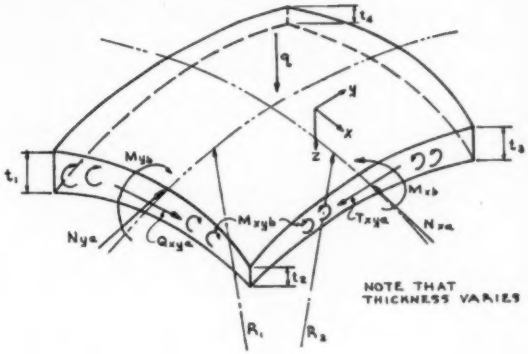


Fig. 2 — Element or finite difference unit

hence

$$\left.\begin{aligned}
&\frac{\partial}{\partial\bar{x}} = R_1\frac{\partial}{\partial x},\ \frac{\partial^2\bar{u}}{\partial\bar{x}^2} = \frac{\partial^2 u}{\partial x^2},\ \frac{\partial^2 v}{\partial\bar{x}\partial\bar{y}} = R_1\frac{\partial^2 v}{\partial x\partial y},\ \frac{\partial^2}{\partial\bar{x}^2} = R_1^2\frac{\partial^2}{\partial x^2},\ \frac{\partial^3\bar{u}}{\partial\bar{x}^3}\\
&= R_1\frac{\partial^3 u}{\partial x^3},\ \frac{\partial^3\bar{u}}{\partial\bar{x}\partial\bar{y}^2} = R_1\frac{\partial^3 u}{\partial x\partial y^2},\ \frac{\partial}{\partial\bar{y}} = R_1\frac{\partial}{\partial y},\ \frac{\partial\bar{u}}{\partial\bar{y}} = \frac{\partial u}{\partial y},\ \frac{\partial^4 w}{\partial\bar{x}^4}\\
&= R_1^3\frac{\partial^4 w}{\partial x^4},\ \frac{\partial^2}{\partial\bar{y}^2} = R_1^2\frac{\partial^2}{\partial y^2},\ \frac{\partial^2\bar{u}}{\partial\bar{y}^2} = R_1^2\frac{\partial^2 u}{\partial y^2},\ \frac{\partial^4 w}{\partial\bar{x}^2\partial\bar{y}^2} = R_1^4\frac{\partial^4 w}{\partial x^2\partial y^2}
\end{aligned}\right\} \quad \text{.....(5)}$$

Therefore, the new nondimensionalized equations are

$$\left.\begin{aligned}
&\frac{\partial^2\bar{u}}{\partial\bar{x}^2} + \tfrac{1}{2}\frac{\partial^2\bar{u}}{\partial\bar{y}^2} + \tfrac{1}{2}\frac{\partial^2\bar{v}}{\partial\bar{x}\partial\bar{y}} - \tfrac{1}{2}\frac{\partial\bar{w}}{\partial\bar{x}} = 0\\
&\frac{\partial^2\bar{v}}{\partial\bar{y}^2} + \tfrac{1}{2}\frac{\partial^2\bar{v}}{\partial\bar{x}^2} + \tfrac{1}{2}\frac{\partial^2\bar{u}}{\partial\bar{x}\partial\bar{y}} - \frac{\partial\bar{w}}{\partial\bar{y}} = 0\\
&\nabla^4\bar{w} + \frac{12R_1^2}{t^2}\left(\frac{5}{4}\bar{w} - \frac{\partial\bar{v}}{\partial\bar{y}} - \tfrac{1}{2}\frac{\partial\bar{v}}{\partial\bar{x}}\right) - \frac{R_1^4 q}{D} = 0
\end{aligned}\right\} \quad \text{.....(6)}$$

The approach in deriving Eq. (6) was to use Southwell's relaxation method.[8] In this system, the shell is considered as broken up into as many parts as one wishes. Each part is of the type shown in Fig. 2 which may be used to represent an infinitesimal element or a finite section. After the shell is broken up into the selected number of pieces, the boundaries of the pieces are imagined to be locked in place. This requires them to be restrained against axial movement, moment, shear, and twisting moment. This fixes all pieces against u, v, and w

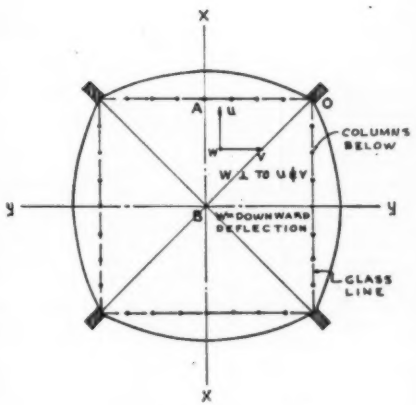
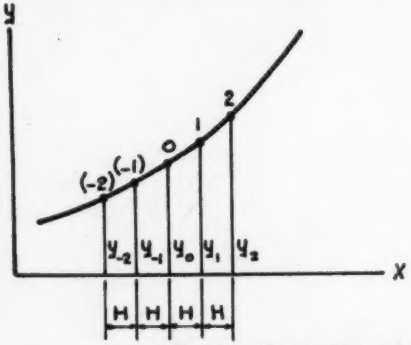


Fig. 3—Plan of shell



$$\left(\frac{dy}{dx}\right)_0 \approx \frac{y_1 - y_{-1}}{2H},$$

$$\left(\frac{d^2y}{dx^2}\right)_0 \approx \frac{y_1 + y_{-1} - 2y_0}{H^2},$$

$$\left(\frac{d^3y}{dx^3}\right)_0 \approx \frac{y_2 - 2y_1 + 2y_{-1} - y_{-2}}{2H^3},$$

$$\left(\frac{d^4y}{dx^4}\right)_0 \approx \frac{y_2 - 4y_1 + 6y_0 - 4y_{-1} + y_{-2}}{H^4}$$

Fig. 4—Illustration of finite differences method

movement (see Fig. 3). This device is analagous to moment distribution where the joints of a frame are first locked against rotation. In fact, moment distribution is a special case of Southwell's relaxation method. Now, just as in moment distribution, we relax our locking devices one by one. However, in the case of a doubly curved element free to move about three axes, both in translation and rotation, no direct straightforward relaxation procedure is possible. Resort must be made to other methods.

For this purpose, the equations were cast in a finite difference form. Here a brief digression on finite differences and liquidation of residuals may save time in what follows. For purposes of illustration, let us take two simple, simultaneous equations such as $4x + 3y = 0$, and $2x - 5y = 26$. Now rewriting the equations we have $4x + 3y = U_1$ and $2x - 5y - 26 = U_2$, where U_1 and U_2 will be known as residuals and should ultimately be reduced to zero, i.e., "liquidated" when the equations are solved. The more closely we approach the proper values of x and y, the smaller the residuals will become. Various methods of operating on equations to liquidate the residuals are available.[3,4] First, we will put our equations into a simple algebraic form similar to the examples used previously. To do this, we make use of finite differences. Here again, to illustrate finite differences, let us take a simple example (see Fig. 4) from which it can be seen that the proc-

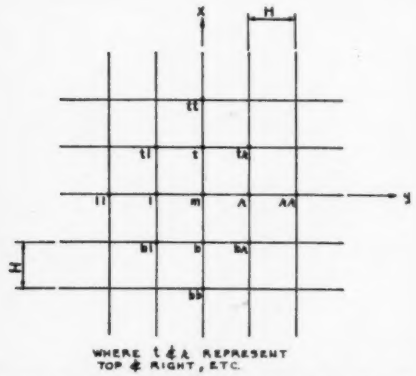


Fig. 5—Finite difference star for fourth order partial

ess of taking successive derivatives consists of "averaging" previous derivatives of a lower order. In the case of partial differential equations, the operator is called a "star"[3,4,8,9] (see Fig. 5). It was seen from Fig. 4 and the previous discussion that five points on a line are needed to get the fourth order differential. Similarly, for the fourth order partial (required because of $\nabla^4\bar{w}$), we need five points in both the x and y direction plus "corner" points for the cross product terms. From the use of our star, we get for the deflections the following expressions:

$$\left.\begin{aligned}
&\frac{\partial\bar{w}}{\partial\bar{x}} = \frac{\bar{w}_l - \bar{w}_b}{2h}, \quad \frac{\partial\bar{w}}{\partial\bar{y}} = \frac{\bar{w}_r - \bar{w}_e}{2h}, \quad \frac{\partial^2\bar{w}}{\partial\bar{x}^2} = \frac{\bar{w}_l - 2\bar{w}_m + \bar{w}_b}{h^2},\\
&\frac{\partial^2\bar{w}}{\partial\bar{y}^2} = \frac{\bar{w}_r - 2\bar{w}_m + \bar{w}_l}{h^2}, \quad \frac{\partial^3\bar{w}}{\partial\bar{x}^3} = \frac{\bar{w}_{ll} - 2\bar{w}_l + 2\bar{w}_b - \bar{w}_{bb}}{2h^3},\\
&\frac{\partial^3\bar{w}}{\partial\bar{y}^3} = \frac{\bar{w}_{rr} - 2\bar{w}_r + 2\bar{w}_b - \bar{w}_{bb}}{2h^3},\\
&\frac{\partial^4\bar{w}}{\partial\bar{x}^4} = \frac{\bar{w}_{ll} - 4\bar{w}_l + 6\bar{w}_m - 4\bar{w}_b + \bar{w}_{bb}}{h^4},\\
&\frac{\partial^4\bar{w}}{\partial\bar{y}^4} = \frac{\bar{w}_{rr} - 4\bar{w}_r + 6\bar{w}_m - 4\bar{w}_l + \bar{w}_{ll}}{h^4}
\end{aligned}\right\} \quad \ldots(7)$$

and the cross-product derivatives are:

$$\frac{\partial^2\bar{w}}{\partial\bar{x}\partial\bar{y}} = \frac{\bar{w}_{lr} - \bar{w}_{rb} + \bar{w}_{bl} - \bar{w}_{ll}}{4h^2},$$

$$\frac{\partial^4\bar{w}}{\partial\bar{x}^2\partial\bar{y}^2} = \frac{\bar{w}_{lr} - 2\bar{w}_r + \bar{w}_{rb} - 2\bar{w}_l + 4\bar{w}_m - 2\bar{w}_b + \bar{w}_{ll} - 2\bar{w}_l + \bar{w}_{bl}}{h^4}$$

Similarly, we obtain expressions for the derivatives of u and v and substituting all of these in Eq. (6) we get

$$\left.\begin{aligned}
&8(\bar{u}_l + \bar{u}_b) + 4(\bar{u}_r + \bar{u}_l) - 24\bar{u}_m\\
&\quad + (\bar{v}_{lr} - \bar{v}_{ll} + \bar{v}_{bl} - \bar{v}_{br}) - 2h(\bar{w}_l - \bar{w}_b) = 0\\
&8(\bar{v}_r + \bar{v}_l) + 4(\bar{v}_l + \bar{v}_b) - 24\bar{v}_m\\
&\quad + (\bar{u}_{lr} - \bar{u}_{ll} + \bar{u}_{bl} - \bar{u}_{br}) - h(\bar{w}_r - \bar{w}_l) = 0\\
&[\bar{w}_{ll} + \bar{w}_{bb} + \bar{w}_{rr} + \bar{w}_{ll} - 8(\bar{w}_l + \bar{w}_b + \bar{w}_r + \bar{w}_l) + 20\bar{w}_m\\
&\quad + 2(\bar{w}_{lr} + \bar{w}_{bl} + \bar{w}_{ll} + \bar{w}_{br})]\\
&\quad + \frac{12h^4R_1^2}{t^3}\left[\frac{5\bar{w}_m}{4} - \frac{(\bar{v}_r - \bar{v}_l)}{2h} - \frac{(\bar{w}_l - \bar{w}_b)}{4h}\right] - \frac{R_1^2qh^4}{D} = 0
\end{aligned}\right\} \quad \ldots(8)$$

Thus far we have only discussed two of the three necessary conditions a structure must satisfy. That is, we have satisfied equilibrium and compatability considerations. It is now necessary to satisfy the boundary conditions. Looking at Fig. 6 we see that AO is supported on columns which prohibit w movement but permit u and v movement; AB permits w and u movements but not v movements due to sym-

metry; OB permits u, v, and w movements. These boundary conditions are expressed by

$$\left.\begin{array}{lll} \text{AO} & \text{AB} & \text{OB} \\ \overline{w}_m = 0,\ \overline{u}_m = \overline{u}_1 & \overline{v}_m = 0,\ \overline{u}_r = \overline{u}_1 & \overline{u}_m = \overline{v}_m,\ \overline{u}_1 = \overline{v}_r \\ \overline{v}_m = \overline{v}_1,\ \overline{w}_1 = -\overline{w}_5 & \overline{v}_r = \overline{v}_1,\ \overline{w}_r = \overline{w}_1 & \overline{w}_5 = \overline{v}_1,\ \overline{v}_5 = \overline{u}_1 \\ & & \overline{v}_1 = \overline{u}_r \end{array}\right\} \quad (9)$$

The above expressions make use of the approximation that radial movement w is equal to vertical movement. This assumption introduces a certain amount of error in the calculations due to the curvature of the shell. In the case of a shell of steep curvature, this would have to be taken into account.

SOLUTION OF THE EQUATIONS

Returning now to Eq. (8), it is necessary that we find a solution for them. At this stage deflections must be assumed for the equations. Since no deflections were known, it was assumed that the u, v, and w deflections were all equal to zero. An attempt at convergence was made by passing the star over the grid. A coarse grid was deliberately chosen so as to expedite the convergence (see Fig. 6). To this end one octant of the shell was divided into 10 x 10 ft pieces. Only an octant need be used for symmetrical loading. All the various locking devices of Southwell's relaxation method were imagined to be in place and then released one by one. As this happened, deflections were pictured to flow to adjacent pieces, then the original piece was again locked into its new position and the star passed on over the grid. The varying thickness of the shell was taken into account as the star moved. This is accomplished by using the average thickness of the four corners of the increment for t in the third equation of Eq. (8). Also, note that t occurs only in the third equation. In this process it greatly expedites convergence if the general character of the deflected shape can be determined. Convergence of three simultaneous equations is always a tedious process because an entire equation must be substituted into another as a residual. The process of moving the star over the grid and smearing the residuals across the boundary would eventually have given convergence regardless of the assumed initial deflected

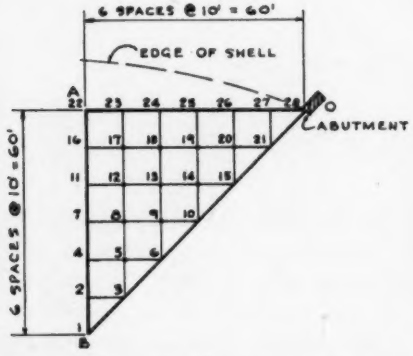


Fig. 6—Coarse grid on octant of shell

shape, i.e., whether zero or any other value. In other words, the system is self-correcting for errors. The errors, too, will eventually be liquidated. However, if the rate of making errors is greater than the rate of convergence, there will be no convergence.

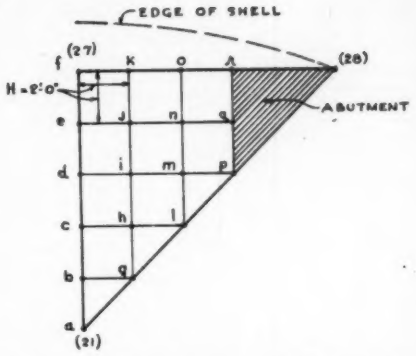


Fig. 7—Grid adjacent to abutment divided into 2 ft squares

After many passes over the grid and no indication of convergence, it was concluded that the convergence was exceedingly slow when starting from $u = v = w = 0$. Hence, it was decided to use a different deflected shape rather than the initial one of $u = v = w = 0$. To this end the shell was divided into 20 ft square pieces with the net intersections indicated as unknowns. This eliminated the need for a star passing over a grid, but resulted in 18 simultaneous equations which were solved. In the interest of brevity, these equations are not reproduced here. The solution of these gave an exceedingly coarse deflection pattern, but it served as a starting point for returning to the liquidation of the residuals by using the star. Hence, these crude deflections in the u, v, w direction were now used as the initial deflections and convergence was then obtained for the 10 ft square increments. The results of this convergence are given in Table 1. It should be noted that Table 1 shows the actual elastic deflections of the shell; hence, it represents the solution of Eq. (3), not of Eq. (6) or (8). From the results of Table 1 it could be seen, as expected, that further refinement of the net was needed near the abutment. Hence, the 10 ft square immediately adjacent to the abutment was divided into 2 ft squares (see Fig. 7). The results of the first convergence were now used as an approximation for convergence of this more refined area. Convergence was once again obtained. It was now believed that the results were sufficiently accurate, although the process could be carried on to any refinement.

STRESS COMPONENTS

Having the deflections, it was now possible to obtain the stresses. Here again the stress components may be derived from the equilibrium and compatibility equations or they may be obtained by extending known equations to our case. By extending the stress component equa-

tions for cylindrical and spherical shells, we get

$$\left.\begin{aligned}
\sigma_{xa} &= \frac{E}{(1-\nu^2)}\left[\frac{\partial u}{\partial x} - \frac{w}{2R_1} + \nu\left(\frac{\partial v}{\partial y} - \frac{w}{R_1}\right)\right] \\
\sigma_{ya} &= \frac{E}{(1-\nu^2)}\left[\frac{\partial v}{\partial y} - \frac{w}{R_1} + \nu\left(\frac{\partial u}{\partial x} - \frac{w}{2R_1}\right)\right] \\
\tau_{xya} &= \frac{E}{2(1+\nu)}\left[\frac{\partial u}{\partial y} + \frac{\partial v}{\partial x}\right] \\
\sigma_{xb} &= \frac{Et}{2(1-\nu^2)}\left[\frac{1}{2R_1}\frac{\partial u}{\partial x} + \frac{\partial^2 w}{\partial x^2} + \nu\left(\frac{1}{R_1}\frac{\partial v}{\partial y} + \frac{\partial^2 w}{\partial y^2}\right)\right] \\
\sigma_{yb} &= \frac{Et}{2(1-\nu^2)}\left[\frac{1}{R_1}\frac{\partial v}{\partial y} + \frac{\partial^2 w}{\partial y^2} + \nu\left(\frac{1}{2R_1}\frac{\partial w}{\partial x} + \frac{\partial^2 w}{\partial x^2}\right)\right] \\
\tau_{xyb} &= \frac{Et}{2(1+\nu)}\left[\frac{1}{2R_1}\frac{\partial u}{\partial y} + \frac{1}{R_1}\frac{\partial v}{\partial x} + \frac{\partial^2 w}{\partial x \partial y}\right]
\end{aligned}\right\} \quad (10)$$

Casting these into finite difference form and using the results of Table 1, we get Table 2 which shows the stresses from the 10 ft square mesh and Table 3 which shows the results of the 2 ft square net near the abutment. By summing up the stresses from Table 3, we can get the load on the abutment, and by subtracting this from the total load, the load on the perimeter columns. The thrust of the shells on the abutments was taken by ordinary reinforcing rods around the perimeter of the building and underground.

BUCKLING

Due to the groining near the abutments, the most likely place for buckling was somewhere in the upper region where both radii approach the R_2 value, i.e., 240 ft. At this location, since $R_1 = R_2 = 240$ ft, we can use the buckling value for a sphere clamped at the edges. We will further assume that the plan diameter of shell participating in buckling is 60 ft. Outside of this circle the groins will be considered to have too much antielastic curvature to buckle. By the classic equations of buckling, the critical stress for a sphere under radial pressure is the same as a cylinder under axial compression, i.e., $\sigma_{CL} = 0.6\,Et/r$. However, the real value of the critical stress is much lower than this. Experimental results have repeatedly shown critical stresses of as low as 10 percent of the classical. By the use of large deflection theory and plasticity effects, results may be obtained which closely approximate experimental values. A spherical shell under radial external pressure will buckle suddenly or "oilcan" by leaping to a lower state of energy at a stress far below that given by the classic value. At present, the classical critical stresses in concrete shells are being reduced by wholly arbitrary ratios which give no assurance of safety and which are not in agreement with one another. A more rational approach was attempted which takes into account the large deflection and plasticity

effects. This approach was performed by using experimental and theoretical work done on metal shells.[5] This is analagous to shell theories of stress distribution in concrete shells. Here the assumption is made that concrete is homogeneous and isotropic, whereas, it is actually neither. By Reference 5 it is seen that the buckling of a shell is a function of the initial roughness, the number of buckle waves that can occur across a shell and the plasticity or creep effect. Of course, these parameters are in addition to the parameters of E, t, and r given in the classical equation. We first need the initial roughness of the shell.

TABLE I — TRUE Δ

	u, in.	v, in.	w, in.
1	0	0	0.820
2	0.0020	0	0.810
3	0.0019	0.0019	0.810
4	0.0045	0	0.810
5	0.0043	0.0018	0.810
6	0.0041	0.0041	0.810
7	0.0076	0	0.810
8	0.0069	0.0016	0.800
9	0.0064	0.0032	0.0790
10	0.0051	0.0051	0.0781
11	0.0098	0	0.0621
12	0.0094	0.0013	0.0768
13	0.0088	0.0026	0.0765
14	0.0074	0.0040	0.0768
15	0.0058	0.0058	0.0714
16	0.0120	0	0.0557
17	0.0121	0.0010	0.0643
18	0.0115	0.0019	0.0694
19	0.0079	0.0028	0.0648
20	0.0096	0.0039	0.0614
21	0.0048	0.0048	0.0528
22	0.0141	0	0
23	0.0141	0.0006	0
24	0.0133	0.0012	0
25	0.0115	0.0019	0
26	0.0090	0.0021	0
27	0.0051	0.0016	0
28	0	0	0

This is defined as the ratio of the amplitude of the imperfection sine wave to the thickness of the shell. For this value it was assumed that the forms could be out of their proper position by one-half the shell thickness, therefore $a_0 = \frac{1}{2}$. This permitted a form deviation of 1½ in., a conservative figure when it is realized that we are only considering the top 60 ft diameter portion. Note the true curves in the form work photos. Reference 5 also shows that experimental and theoretical results have shown that for a value of $d^2/(rt)$ between 42 and 85, we get two buckles equally spaced about the pole. In our case $d^2/(rt) = 60^2 \times 4/240 = 60$, hence, $n = 2$. From knowing that we may expect two

TABLE 2 — STRESS RESULTS OF MESH SHOWN IN FIG. 6

	σ_{xa}, psi	σ_{ya}, psi	σ_{xb}, psi	σ_{yb}, psi	τ_{xya}, psi	τ_{xyb}, psi
1	−50	−160	0	0	0	0
2	−50	−150	− 3	−11	0	0
3	−60	−160	− 4	− 4	− 4	+1
4	−35	−110	0	− 9	0	0
5	−50	−150	− 4	− 6	− 6	−3
6	−58	−153	− 8	− 1	−18	+1
7	−24	−132	− 5	−16	0	0
8	−50	−160	− 3	− 5	−14	−1
9	−58	−165	+ 2	− 1	−27	+1
10	−65	−152	− 2	− 8	−46	0
11	−22	−169	0	−10	0	0
12	−35	−156	+ 3	− 3	−13	0
13	−47	−172	− 6	− 5	−27	−1
14	−51	−164	−10	− 8	−42	−2
15	−60	−144	− 4	−19	−58	0
16	−24	−122	−30	+ 8	0	0
17	−36	−149	−48	+15	−10	−3
18	−46	−166	−53	+12	−30	−1
19	−25	−145	−43	+23	−50	+1
20	−52	−139	−44	+38	−54	+3
21	See Table 3					
22	+34	+ 25	0	0	0	0
23	+36	+ 25	0	0	− 8	0
24	+27	+ 25	0	0	−25	0
25	+29	+ 19	0	0	−44	0
26	+22	− 6	0	0	−68	0
27	See Table 3					

half-waves across the shell and that the initial imperfection ratio = ½, we find from the given equation

$$U_o = a_o \left(\frac{2nt}{d}\right)^2 = \frac{1}{2}\left(\frac{4}{60 \times 4}\right)^2 = \frac{1}{7200} \quad \text{(11)}$$

From this $U_0R_2/t = 240 \times 4/7200 = 0.13$. By entering Fig. 45 of Reference 5 we find that

$$\frac{\sigma_{CR}}{\sigma_{CL}} = 0.40 \quad \text{(12)}$$

We have not yet taken into account the effect of plasticity. Here we will assume as has frequently been done in other plasticity considerations of concrete that $E_s = 3E_t = E$. From this and the knowledge that

$$\nu = \nu_p - \frac{(\nu_p - \nu_e)}{E} E_s \quad \text{(13)}$$

we get $\nu = \nu_e$ (which was postulated in the nomenclature). Reference 5 gives the plasticity effect as

$$\eta = \frac{E_s}{E}\sqrt{\frac{(1-\nu_e^2)}{(1-\nu^2)}\,\frac{Et}{E_s}} \quad \text{(14)}$$

TABLE 3 — STRESS RESULTS OF MESH SHOWN IN FIG. 7

	σ_{xa}, psi	σ_{ya}, psi	σ_{xb}, psi	σ_{yb}, psi	τ_{xya}, psi	τ_{xyb}, psi
a	−131	−204	− 57	− 57	− 83	+ 51
b	− 66	−177	− 8	− 54	−124	− 18
c	− 64	−190	− 66	−114	−133	− 28
d	− 53	−163	− 38	−146	−113	− 23
e	− 32	−131	−394	−232	− 96	+ 55
f	+ 2	− 40	0	0	− 88	0
g	−101	−163	− 58	− 58	−192	− 57
h	− 93	−200	− 44	− 72	−208	− 67
i	− 62	−176	− 30	− 86	−183	− 25
j	− 38	−134	−136	− 20	−163	−126
k	− 12	− 79	0	0	−154	0
l	−123	−159	− 18	− 18	−204	− 63
m	− 27	−184	+ 80	− 4	−146	+ 28
n	− 26	−134	+ 2	+ 90	−158	+148
o	− 14	− 98	0	0	−152	0
p	−201	−201	+528	+528	−267	+ 69
q	− 23	−150	+ 32	+214	−133	0
r	− 16	−108	0	0	−133	0

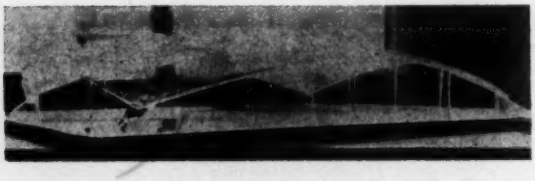


Fig. 8—View of model

which gives $\eta = 0.58$. Multiplying the results of Eq. (12) and (14) together, we get a factor of 0.23. In other words, the critical buckling load is 23 percent of the classical value. This is a value which is in good agreement with test results. Therefore, we find

$$\sigma_{cr} = \frac{0.23 \times 0.60 \times 3 \times 10^6}{4 \times 240} = 420 \text{ psi}$$

When this is compared with Point 8 in Table 1, we find a factor of safety of buckling of $420/160 = 2.60$.

MODEL ANALYSIS

In addition to the theoretical analysis, a model analysis was made. The model is shown in Fig. 8, 9, and 10. The scale is 1:60. The model was constructed of plexiglass and was made by suction of the plexiglass into a plaster of Paris mold in an oven. Various strain gages and rosettes were connected to the upper and lower surface of the shell and wired up to the Wheatstone bridge as shown in Fig. 9.

Loading on the model was accomplished by bags of lead shot placed in the following loading conditions:

Case I—Full span load of 180 lb total

Case II—Half span load of 100 lb total

Case III—Full span load of 90 lb with columns cut

Case IV—Stresses produced in the unloaded shell by the stabilizing columns (i.e., residual stresses produced by the columns)

Due to a limitation of funds for the experimental program, the correlation between model stresses and theoretical analysis in general was poor. The theoretical analysis yielded stresses which appeared more

Fig. 9—View of model with strain gages in place

Fig. 10—Forming of toroid

logical than the model analysis. However, some general conclusions could be drawn from the model test. These were:

1. The stresses in the shell under all conditions of loading are low. This compares with the theoretical analysis results.
2. Stresses caused by cutting the columns were larger than those caused by dead and live load.
3. Eliminating the columns would cause large deflections of the edge.

The last conclusion became particularly significant in the light of what happened during construction.

CONSTRUCTION

Two conditions arose on the job which caused variations from the theoretical stresses. Two years elapsed between the analysis and the start of construction of the shell. During this period the requirements of the owner changed and the dimensions of the shell were changed from 120 to 128 ft square. This was accomplished by increasing all dimensions proportionately. The second change occurred due to an architectural requirement that the mullions be made nonload supporting. While it was known that this would increase certain stresses, still it was believed that they would remain within tolerable limits. Hence, the mullions were cut free. This was accomplished by using a telescoping joint at the top of the mullion. As predicted by the model analysis, this caused the high point of the edge to drop about 3 in. while the quarter point of the edge rose about 1 in. Meanwhile, the center of the shell had dropped about 2 in. instead of the ⅞ in. given by Table 1. This latter deflection was anticipated in the design as it was expected that the elastic deflections would not represent the actual deflections. Readings of the edge deflection were made for several months. During this time it was observed that the center of the edge

Fig. 11—Surface forming of toroid

rose and fell approximately ½ in. per day due to temperature changes. It might have been possible to partially rationalize the measured deflections against a new set of theoretical deflections using the new edge conditions in the analysis. However, this would have required solving Eq. (8). The higher deflections are caused by the shear and axial stresses in the y-direction.

The shell was formed completely and cast as one monolithic piece (see Fig. 10, 11, and 12). This was not an engineering requirement, but was the choice of the contractor. The contractor was given the length of all the shores as calculated from the geometry of a torus. This equation and the equation of the intersecting line of the two tori were derived on Fig. 1. The contractor then ran curved members over the tops of the shores to generate the surface of the shell.

The reinforcing steel and concrete are quite minimal as can be seen by Fig. 13. Since only the stresses near the abutment are high only the heavy radial bars fanning out from the abutment as shown in the inset of Fig. 13 are placed for maximum efficiency. The choice of the reinforcing pattern for the bulk of the shell was made for simplicity of placing. The design strength of the concrete was $f_c' = 4000$ psi at 28 days. The shell is 3 in. thick in the center. The thickness was varied on the upper surface of the shell. This thickening was done in accordance with contour lines (see Fig. 14). The unsupported edge which has no stiffening ribs is 5 in. thick and spans 128 ft. The abutments are cast monolithically with the shell, thereby eliminating costly hinges.

Fig. 12 — View of toroid during concreting

Furthermore, the shell gains additional strength from the moment connections at the supports. The forms were stripped in a spiral pattern starting at the center. Due to the double curvature, large sections of forms would "hang up" on the underside of the shell and not drop until their weight got large enough to break them free. This caused unbalanced loads on the shell and impact loads when the forms dropped. However, the shell did not suffer. The shell after being in place about 2½ years exhibited no distress.

WAVE FORM SHELLS

The wave form shells shown in Fig. 11, and 14 are also of some interest. The shells are 2 in. thick. The chord length is 20 ft and the axial length between columns varies from 20 to 30 ft; however, the total axial length was between 60 and 120 ft. They have no stiffening

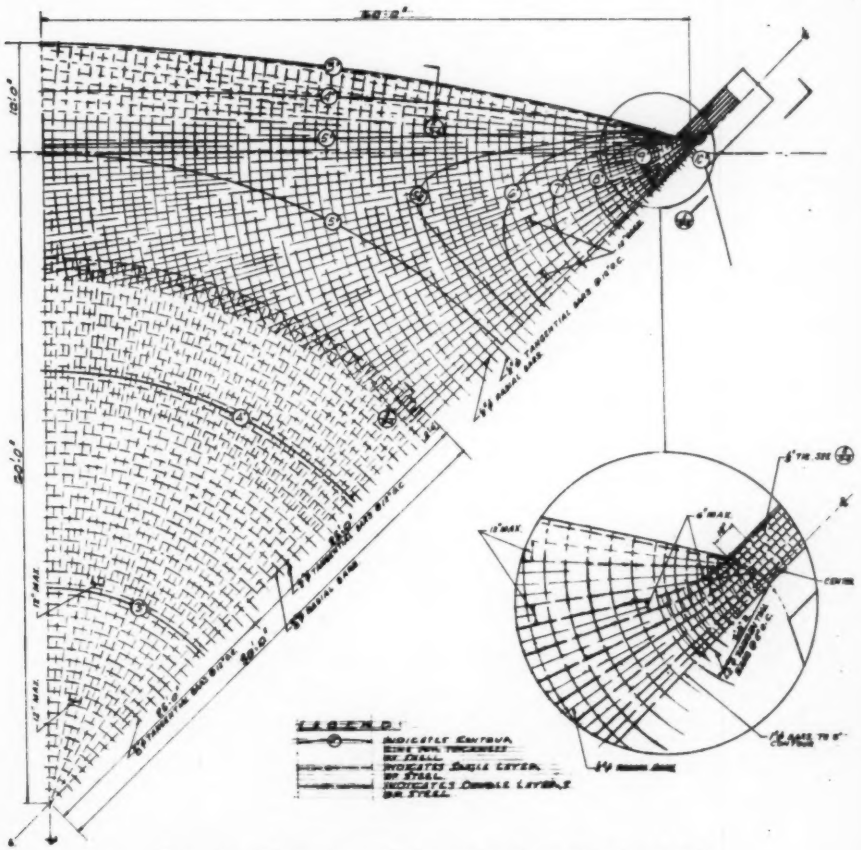


Fig. 13—Plan view of the torus (one-eighth scale)

Fig. 14—General view of shopping center

ribs but only a shear block over the columns to prevent punching shear failure. The columns are located at the lowpoint of the corrugation. There are no tie rods, the thrust being taken by the columns. These shells were built by moving forms lengthwise along their axes. Joints were permitted at any location. Thus, they were built in a manner similar to tunnels. By using this method, it was possible to strip forms in five days.

CONCLUSIONS

Southwell's Relaxation method applied to the general equations of a doubly curved shell will solve any shell for any condition of loading. It would have been possible to do half span loadings, or wind or seismic analysis by this method. However, the method may become exceedingly tiresome. With more accurate knowledge of all the stresses, including bending stresses, it is possible to eliminate hinges in most shells and edge beams in some of them. However, it should be emphasized that each shell is a special case and requires individual investigation. Also, less reinforcing steel is needed if the engineer is more confident of the stresses.

Buckling should be determined by a more rational approach than the present system of arbitrary reductions. To this end, available theoretical and experimental data should be applied to concrete shells. Calculations for buckling should take into account initial eccentricity, plasticity effects and large deflection theory. Otherwise, we are merely guessing. The use of strain gage model analysis is helpful but adequate models are expensive and many strain gages must be used to get good results.

ORGANIZATION

The architects on the project were Wimberly and Cook, Honolulu, the contractors were Nordic Construction Co., Honolulu. The author was the structural engineer.

ACKNOWLEDGMENT

The use of the relaxation procedure for stress analysis was suggested by Jackson Wignot of Lockheed Aircraft Corp. The author wishes to express his appreciation to Mr. Wignot for his assistance in completing this work.

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American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

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Aplicación de la teoría general de los cascos

Las ecuaciones diferenciales generales de un casco se presentan con un método de solución de las mismas. Los términos para el momento de flexión y de torsión se incluyen. Las condiciones de corde se consideran en la solución. El casco no tiene ni nervaduras ni bisagras. Se presenta también un concepto más racional del pandeo de los cascos de concreto, teniéndose en cuenta la teoríá de gran flecha sobre los efectos de pandeo y de plasticidad. Se discuten la construcción y el comportamiento del casco.

Application de la théorie générale des coquilles

Des équations générales différentielles d'une coquille sont présentées avec une méthode pour leur solution. Les termes du moment de la flexion et de la torsion sont y compris. Les conditions limites sont considérées dans la solution. La coquille n'a ni nervures ni charnières. Aussi, une approche plus rationnelle à la déformation de coquilles de béton est présentée avec considération de la théorie de grande déflexion des effets de la déformation et de la plasticité. La structure et le comportement de la coquille sont discutés.

Anwendung der allgemeinen Schalentheorie

Allgemeine Differenzialgleichungen einer Schale werden zusammen mit einer Loesungsmethode fuer dieselben dargestellt. Ausdruecke fuer das Biege- und Drehmoment sind auch einbezogen. Die Grenzbedingungen werden in der Loesung beruecksichtigt. Die Schale hat weder Rippen noch Gelenke. Ebenso wird eine rationellere Betrachtungsweise des Verwerfens von Betonschalen gegeben, mit besonderer Beruecksichtigung der Theorie von grossen Durchbiegungen von Verwerfungs- und Plastizitaetswirkungen. Die Konstruktion und das Verhalten der Schalen werden besprochen.

Title No. 58-6

Fatigue Properties of Lightweight Aggregate Concrete

By WARREN H. GRAY, JOHN F. McLAUGHLIN, and JOHN D. ANTRIM

Fatigue tests were conducted on two different lightweight aggregate concretes, one proportioned for a high strength and the other for a low strength. Specimens of approximately the same age were tested at stress levels of 40, 50, 60, 70, and 80 percent of the ultimate static compressive strength of the respective mixes. Within the limits of the investigation, the fatigue behavior of high strength lightweight concrete was similar to that of low strength lightweight concrete. In addition, the fatigue behavior of the lightweight aggregate concrete appears to be similar to that found for a normal weight concrete in a previous study.

■ LIGHTWEIGHT AGGREGATE CONCRETE has proved itself to be a useful structural material; its uses and applications are becoming more numerous as engineers learn more about it. The literature contains the results of numerous research projects carried out to determine the properties of the various lightweight aggregates available and some on strength, volume change, and thermal properties of the finished concrete. Published information on the fatigue resistance of lightweight concrete is scant.

The resistance of a material to repeated loading is obviously an important factor in design. A working stress must be chosen such that the structure will not fail from repeated applications of loads below the ultimate strength. Knowledge of the relationship between the number of cycles to failure and applied stress is essential. Many studies have been conducted to determine the fatigue resistance of normal weight concrete[1-5] but more information is needed on the behavior of lightweight concrete subjected to this type of loading. The work reported in this paper had as its major objective, the establishment and comparison of the S-N relationship of two lightweight aggregate concretes, and the comparison of these two relationships with two curves previously established for a normal weight concrete.

TESTING PROGRAM

The testing program was divided into two parts. In the first part tests were conducted on a low strength concrete which will be referred to as the LL series. The second part consisted of conducting tests on a high strength concrete which will be referred to as the HL series. Each series was made up of five batches of concrete and from each batch 30 cylinders 3 in. in diameter and 6 in. long were cast.

Materials and proportioning

The fine and coarse aggregate used in this study was an expanded shale produced in a rotary kiln. Type I portland cement from a single clinker batch was used in both mixes and an air-entraining agent was added at the mixer. The mixes were designed in accordance with the "Recommended Practice for Selecting Proportions for Structural Lightweight Concrete (ACI 613A-59),"[7] except that the specific gravity factor was not computed.

The average strength of the low strength concrete was 3700 psi with a slump of 2½ in. and an air content of 7 percent. A cement factor of 5.8 bags per cu yd was used. The average strength of the high strength concrete was 6200 psi with a slump of 2½ in. and an air content of 6½ percent. A cement factor of 9.6 bags per cu yd was used. The low strength concrete contained 60 percent fine aggregate by weight of total aggregate and the high strength concrete contained 65 percent fine aggregate by weight of total aggregate.

Specimens were cured in water for 28 days in accordance with ASTM C 129-52T, after which, they were placed in an oven to dry for 4 to 5 days at approximately

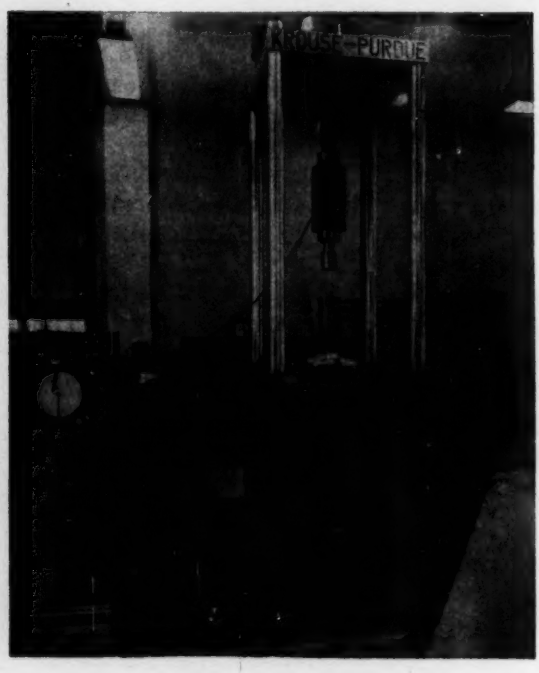

Fig. 1 — Krouse-Purdue fatigue machine

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105 C. The object of drying was to prevent further hydration during the period of fatigue testing. After the specimens were removed from the drying oven, they were allowed to cool for 24 hr before capping. Caps were then placed on each end of all cylinders with a sulfur capping compound and the cylinders were stored at room temperature until testing.

Testing methods

Shortly before fatigue testing, static compression tests were conducted on five randomly chosen specimens from each batch to estimate the ultimate strength of the batch. The static tests served as an estimate of the average batch strengths and the stress levels used in fatigue testing were determined from these results.

Since at least 2 weeks were required to complete the fatigue testing on any one batch, static tests were conducted after fatigue testing on the remaining specimens in those batches that had enough specimens left to insure a reasonable estimate of the batch strength. These tests were to detect any gain in strength which took place during the time of testing.

Eight specimens were chosen from each batch for testing under a pulsating load at stress levels of 40, 50, 60, 70, and 80 percent of the estimated static compressive strength of the batch. Specimens were tested in direct compression only and a minimum stress of between 70 and 170 psi was maintained on all specimens to eliminate any possibility of impact. In some cases, where power failures or testing machine breakdowns occurred, it was not possi-

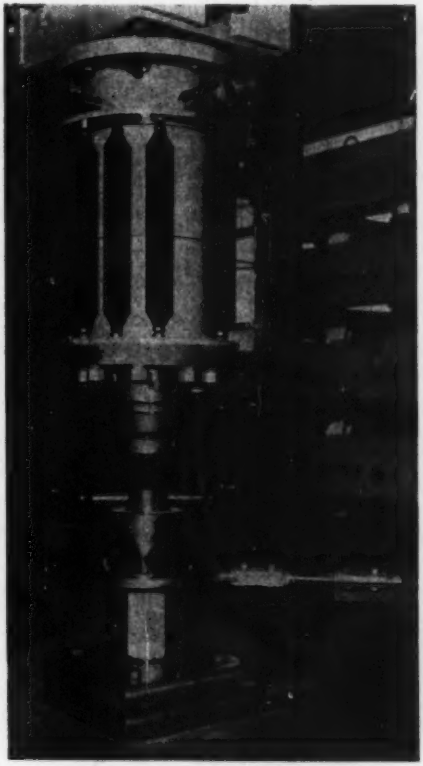

Fig. 2—Loading jack

ble to test specimens from each batch at all stress levels. If testing was interrupted, the specimen was discarded and the test was rerun on a new specimen.

Two testing machines were employed in the fatigue testing program, the Krouse-Purdue machine, and an Amsler machine. The Krouse-Purdue machine is of the constant deflection type which derives its force from hydraulic pressures acting on a piston (see Fig. 1). This machine has a capacity of ±60,000 lb and operates at 1000 cycles per min.

The Amsler fatigue testing machine is based on the same principle as the Krouse-Purdue machine except that the loading jack is separated from the pulsator. Loads are transmitted by hydraulic pressure through the tubing to the load jack which is mounted on a specially built frame. The jack develops its force by pushing against the frame and the concrete specimen which is placed on a bearing plate on the floor as shown in Fig. 2. This machine has a capacity of 110,000 lb and was operated at a speed of 500 cycles per min.

Since the Krouse-Purdue machine tested at a rate of 1000 cycles per min and the Amsler machine was used at the maximum speed of 500 cycles per min, tests were conducted to see if this variation would affect the fatigue test results. Batch HL 1 was selected for this purpose with one-half of the specimens being tested in each machine. All of these specimens were tested at the 80 percent stress level.

DISCUSSION OF RESULTS

The discussion of the results of this study has been divided into three parts. The first part is concerned with physical properties of the plastic concrete and the static tests on the hardened concrete. The discussion of the fatigue testing data is presented in the second part and the third part is concerned with the comparison of the results of this study with the results of a previous study on normal weight concrete.

Analysis of mix data

Since each series consisted of a group of five separately mixed batches, estimates of the properties of the concrete for each series were made from the properties measured on individual batches. The batch properties of slump, air content, and static ultimate strength are summarized in Tables 1 and 2.

The analysis of variance was used to test for any significant difference between the average batch strengths.[8] Data from the low-strength concrete and the high-strength concrete were analyzed separately, and then the observed strengths for the two series were compared. All comparisons were made at the 5 percent significance level and the following results were found:

1. The average strength of the low strength series was 3760 psi with a standard error of 294 psi, and the average strength of the high strength series was 6260 psi with a standard error of 329 psi.

2. The variances of the batch strengths were not significantly different in either the high strength or low strength concrete.

3. The batch means were significantly different in the low strength series but were not significantly different in the high strength series.

4. The total variance of the low strength series was not significantly different from the total variance of the high strength series.

5. The average air content of the low strength series is 6.8 percent with a coefficient of variation of 9 percent, and the average air content of the high strength series was 6.5 percent with a coefficient of variation of 5 percent.

6. There was no significant change in ultimate strength during the time of fatigue testing.

These results indicated that two distinct populations were being tested and further that differences in concrete properties among batches were not statistically significant except for the case of strength in the LL series. Examination of the means shows a range of 560 psi in this series which was considered low enough to ignore for the purposes of fatigue testing. Therefore, specimens were chosen for fatigue testing at random from the entire number prepared and no batch effect was measured.

TABLE 1 — PHYSICAL PROPERTIES OF CONCRETE LOW STRENGTH (LL) MIX

Batch designation	Air content, percent	Slump, in.	Average ultimate strength	
			After oven drying,* psi	After fatigue testing,† psi
LL 1	6.5	2⅝	3500	3280
LL 2	6.1	2⅜	3820	None tested‡
LL 3	7.6	3¼	4060	3900
LL 4	7.5	3	3880	None tested
LL 5	6.2	1½	3530	None tested

*Specimen tested at 34 days.
†Specimen age ranged from 51 to 58 days.
‡All specimens were used in fatigue testing.

TABLE 2 — PHYSICAL PROPERTIES OF CONCRETE HIGH STRENGTH (HL) MIX

Batch designation	Air content, percent	Slump, in.	Average ultimate strength	
			After oven drying,* psi	After fatigue testing,† psi
HL 1	6.8	2⅜	5130	None tested‡
HL 2	6.6	2⅛	6310	None tested
HL 3	6.5	2⅜	6360	None tested
HL 4	6.3	1⅞	6360	6180
HL 5	6.8	3	6260	None tested
HL 6	7.0	2¾	6010	5790

*Specimen age ranged from 44 to 69 days.
†Specimen age ranged from 84 to 94 days.
‡All specimens were used in fatigue testing.

Fatigue test results

The results of the fatigue data are summarized in Tables 3 and 4. These data are shown in graphic form in Fig. 3 and 4, the S-N diagrams, where stress level in percent is plotted against cycles to failure on a logarithmic scale. As is usual with most fatigue data, there is considerable variation evident, the presence of which makes the interpretation somewhat difficult. However, the purposes of this investigation would be satisfied if curves were fitted to the data from the two mixes, compared to each other, and compared to a comparable curve developed from normal weight concrete.

Since 10 million cycles was set as the upper limit of stress repetitions and many specimens tested at various stress levels endured this number, it was not possible to fit a curve by statistical means over the entire range of the data. For example, in the case of the low strength concrete, all of the specimens tested at the 40 percent stress level and all except one specimen tested at the 50 percent level endured 10 million repetitions of loading without failure. Specimen 2 of batch LL 3 had endured 9.2 million cycles when a power failure stopped the machine and no doubt would have gone to 10 million repetitions.

If the assumption that Specimen 2 of batch LL 3 endured 10 million cycles without failure is accepted, the data at the 50 percent stress level

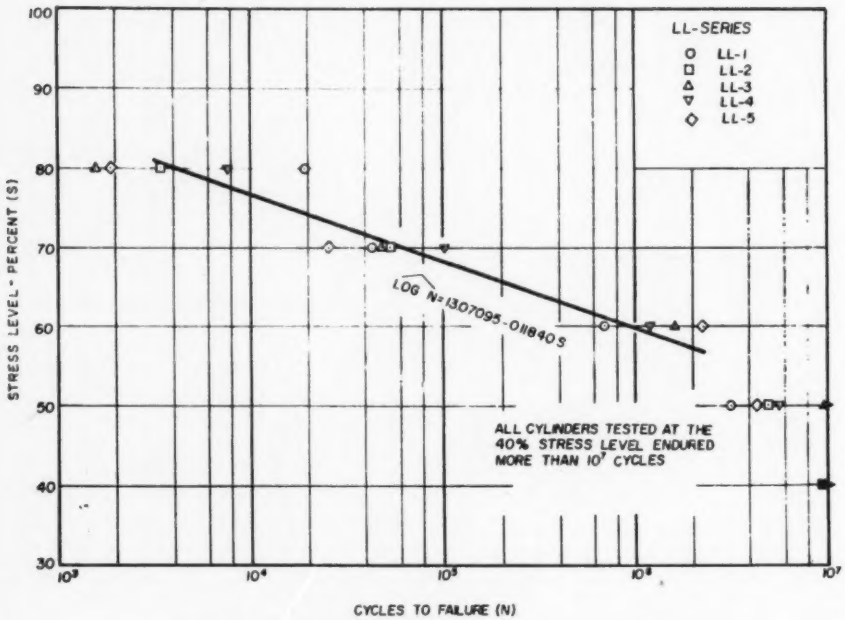

Fig. 3—S-N diagram for low strength concrete

is incomplete. Using only the four specimens that failed at the 50 percent stress level, when actually five were tested, would introduce a bias into the statistical interpretation of the fatigue data. For this reason a better approximation can be made of the S-N relationship if only the data at the 60, 70, and 80 percent stress levels are included in the calculations.

In the case of the high strength concrete, the first two specimens tested at the 40 percent stress level endured the maximum of 10 million cycles of loading without failure. Since all of the specimens of the LL series tested at the 40 percent stress level endured 10 million repetitions of loading, it was felt that no further information could be gained from testing the remaining scheduled specimens of the HL series at the 40 percent stress level. At the 50 percent stress level only two specimens failed before they had endured 10 million cycles of loading so for purposes of regression analysis, data collected at the 50 percent level was not to be used to interpret the S-N relationship of the HL series.

TABLE 3 — FATIGUE TEST RESULTS
LOW STRENGTH CONCRETE

Batch designation	Specimen No.	Maximum fatigue load, lb	Minimum fatigue load, lb	No. of stress cycles endured
LL 1	1	9,900 (40)*	900 (3.6)*	10,304,600†
	2	12,400 (50)	800 (3.2)	3,147,600
	3	14,800 (60)	700 (2.8)	688,700
	4	17,300 (70)	900 (3.6)	43,000
	5	19,800 (80)	800 (3.2)	19,100
LL 2	1	10,800 (40)	900 (3.3)	10,005,400†
	2	13,500 (50)	800 (3.0)	4,926,400
	3	16,200 (60)	500 (1.9)	396,600
	4	18,900 (70)	800 (3.0)	52,800
	5	21,600 (80)	800 (3.0)	3,500
LL 3	1	11,500 (40)	700 (2.4)	10,464,100†
	2	14,300 (50)	500 (1.7)	9,204,100†
	3	17,200 (60)	900 (3.1)	1,610,000
	4	20,100 (70)	800 (2.7)	51,400
	5	23,000 (80)	500 (1.7)	1,600
LL 4	1	11,000 (40)	1000 (3.6)	10,418,100†
	2	13,700 (50)	1000 (4.0)	5,673,300
	3	16,500 (60)	900 (3.3)	1,217,000
	4	19,200 (70)	800 (2.9)	105,100
	5	22,000 (80)	900 (3.3)	8,600
LL 5	1	10,000 (40)	1000 (4.0)	11,723,300†
	2	12,500 (50)	1000 (4.0)	4,628,100
	3	15,000 (60)	1000 (4.0)	2,262,500
	4	17,500 (70)	900 (3.6)	26,300
	5	19,900 (80)	1100 (4.3)	1,900

*Figure in parentheses is the dynamic load expressed as a percentage of the average ultimate strength of the batch.
†Specimen had not failed when the test was stopped.

Again, the S-N relationship was computed from data collected at the 60, 70, and 80 percent stress levels and should only be applied within this range.

Comparison of the two mixes—Since the form of the distribution of fatigue data was unknown, it was felt that a test should be employed to determine whether or not the two sets of data (LL and HL) represent populations having the same frequency distribution. A method called "Runs" was chosen for this purpose.[9] When this test was used to test the data that was applicable, it indicated that, at the 5 percent significance level, there was no reason to believe that the data at each stress level did not represent the same population.

S-N diagrams—A semi-log coordinate system was used to plot the S-N relationship. The data collected from the LL series are plotted in Fig. 3 and the data collected from the HL series are plotted in Fig. 4.

When the data from the HL series were plotted there appeared to be two separate and distinct relationships. The concrete of batches HL 5 and HL 6 seemed to have different fatigue characteristics than the concrete of batches HL 2, HL 3, and HL 4. The reason for this separation of data is unknown. Three curves were fitted to the fatigue test data, one for the LL series and two for the HL series. For purposes of cal-

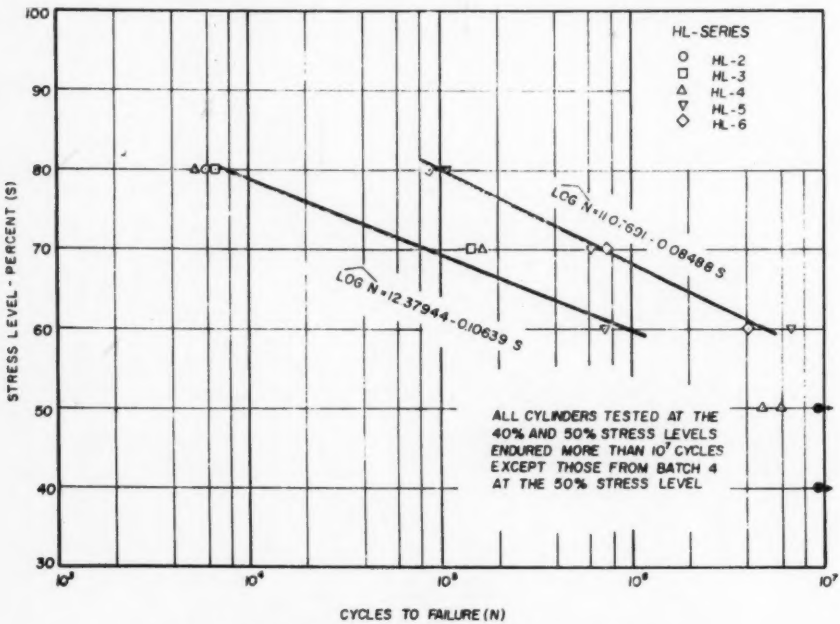

Fig. 4—S-N diagram for high strength concrete

culation, the LL series will be referred to as Sample I. Batches HL 2, HL 3, and HL 4 will be referred to as Sample II and batches HL 5 and HL 6 will be referred to as Sample III. Curves were fitted to each set of data by the method of least squares.

The least squares linear regression equation obtained from the data of Sample I (LL series) is

$$\widehat{\log N} = 13.077 - 0.118\ S$$

where N is the number of cycles to failure and S is the stress expressed as a percentage of the static ultimate stress. This equation can be used to estimate N only when the value S lies between 60 and 80 percent because this is the range of S used in the calculations.

TABLE 4 — FATIGUE TEST RESULTS
HIGH STRENGTH CONCRETE

Batch designation	Specimen No.	Maximum fatigue load, lb	Minimum fatigue load, lb	Number of stress cycles endured
HL 2	1	17,800 (40)*	1000 (2.2)*	10,458,700†
	2	22,200 (50)	1200 (2.7)	10,620,300†
	3	26,700 (60)	1100 (2.5)	720,500
	4‡			
	5	35,500 (80)	1200 (2.7)	6,100
HL 3	1	18,000 (40)	1100 (2.4)	10,216,500†
	2	22,500 (50)	900 (2.0)	10,474,400†
	3	27,000 (60)	1000 (2.2)	9,673,500
	4	31,500 (70)	1000 (2.2)	150,900
	5	36,000 (80)	1000 (2.2)	6,600
HL 4	1‡			
	2	22,500 (50)	1000 (2.2)	4,751,300
	3	22,500 (50)	1200 (2.7)	5,957,200
	4	27,000 (60)	1000 (2.2)	736,400
	5	31,000 (70)	1000 (2.2)	166,100
	6	36,000 (80)	1000 (2.2)	5,400
HL 5	1‡			
	2	22,200 (50)	1300 (2.9)	10,499,400†
	3	26,600 (60)	1100 (2.5)	6,737,500
	4	31,000 (70)	1100 (2.5)	611,900
	5	35,400 (80)	1100 (2.5)	110,000
HL 6	1‡			
	2	21,200 (50)	1200 (2.8)	10,196,100†
	3	25,400 (60)	1100 (2.6)	4,116,700
	4	29,700 (70)	1100 (2.6)	753,600
	5	33,900 (80)	1100 (2.6)	86,300

*Figure in parentheses is the dynamic load expressed as a percentage of the average ultimate strength of the batch.
†Specimen had not failed when the test was stopped.
‡Specimen not tested.

The least squares linear regression equation obtained from the data of Sample II is

$$\widehat{\log N} = 12.379 - 0.106\ S$$

The use of this equation is also limited to values of S between 60 and 80 percent.

The least square linear regression equation obtained from the data of Sample III is

$$\widehat{\log N} = 11.077 - 0.085\ S$$

The use of this equation is limited to values of S between 60 and 80 percent.

Calculations for correlation coefficients were also included in the linear regression analyses. For all three samples there is obviously a high degree of association between cycles to failure and stress level. The correlation coefficients were found to be -0.952, -0.982, and -0.956 for Samples I, II, and III, respectively. Comparison tests showed that there is no reason to believe that these correlation coefficients are different. Hence, the degree of association between the stress level and the number of cycles to failure can be assumed to be the same for all three samples.

An analysis of covariance was used to determine whether the slopes of the three regression curves were significantly different. The computation form for this analysis is described in Reference 8. The results of this test indicate that there is no reason to believe that the slopes of the three regression equations are different.

A two-way analysis of variance test was conducted on the combined data of all three samples to see if there was any difference in the intercepts of the regression equation. This analysis indicates that there is a significant difference between intercepts. A Newman-Kuels[8] sequential range test showed that the intercept of Sample III was different than the intercepts of the other two samples.

Prediction intervals were also calculated for the three S-N relationships. When the prediction intervals were compared it appeared that the data of Sample III were different than the data of Samples I and II.

The relationship between the stress level and the number of cycles to failure is represented most clearly by the slope of the linear regression equation. The results of the analysis of slopes suggest that the data of all three samples have the same slope and therefore suggest that all three samples have the same S-N relationship. The prediction intervals tend to weaken the conclusion that all three samples have the same S-N relationship but this test is not as strong as the analysis of slopes because individual errors rather than a total combined error

is used. It is, therefore, felt that the strongest and most likely conclusion that can be drawn from this analysis is that the fatigue properties of all concrete tested in this study are the same regardless of differences in the static properties.

Effect of rate of load application—Nine specimens were tested in each machine (Krouse-Purdue and Amsler) at the 80 percent stress level and a *t*-test was made on the means obtained. Specimens tested in the Krouse-Purdue machine were tested at a rate of 1000 cycles per min and those tested in the Amsler machine were tested at a rate of 500 cycles per min. The statistical analysis of these fatigue tests indicated strongly that, between the limits of 500 and 1000 cycles per min, the rate of load application has no effect on the fatigue properties of the lightweight concrete used in this study.

Summary of fatigue test results—The results obtained from the analysis of fatigue test data may be summarized as follows:

1. The linear regression equation of the S-N relationships for the low-strength concrete for S between 60 and 80 percent is

$$\widehat{\log N} = 13.077 - 0.118\ S$$

2. The data of the high strength series appeared to form two separate S-N relationships. The linear regression equations of these two S-N relationships for S between 60 and 80 percent are

$$\widehat{\log N} = 12.379 - 0.106\ S$$

and

$$\widehat{\log N} = 11.077 - 0.085\ S$$

3. There is no reason to believe that a difference exists between the fatigue properties of low strength and the high strength concrete.

4. The S-N diagrams appear to show no fatigue limit for lightweight aggregate in the neighborhood of 10 million repetitions of loading.

5. There is no reason to believe, from the results obtained on the 18 specimens tested, that the rate of load application has an effect on the fatigue properties of lightweight aggregate concrete when the rate of load application lies between 500 and 1000 cycles per min.

Comparison of lightweight concrete with normal weight concrete

In May 1959 the results of a study comparing the fatigue properties of air-entrained concrete with the fatigue properties of non-air-entrained concrete was reported.[5] That study was made in the same laboratory as the present study. The testing programs were nearly identical except

for the aggregates used. The coarse aggregate used in the 1959 study was a crushed limestone from central Indiana and the fine aggregate was from a local river terrace deposit.

A comparison was made to determine whether the fatigue properties of lightweight concrete are different from the fatigue properties of normal weight concrete observed in the earlier study. Statistical tests indicated that at the 5 percent significance level there is no significant difference in the slopes of the S-N curves for lightweight and normal weight concrete. Since the slope is the property which best defines the S-N relationship, it seems reasonable to conclude that the fatigue properties of the concrete used in the present study do not differ from the fatigue properties of normal concrete.

A test for difference in correlation coefficients indicated that there is a significant difference between the correlation coefficients of the five S-N relationships tested. A comparison of these coefficients shows that the correlation coefficient for the non-air-entrained series reported in Reference 5 is about 30 percent lower than for the other four values compared. This comparison indicates that the degree of association between stress level and number of cycles to failure is much higher for concrete containing entrained air.

CONCLUSIONS

The following conclusions can be drawn from the fatigue data collected in this study. These conclusions are based on the fatigue testing of forty-seven 3 x 6 in. lightweight aggregate concrete specimens tested in direct compression at a rate of loading which varied from 1000 to 500 cycles per min.

1. The fatigue properties of lightweight aggregate concrete are not significantly different over large variations in strength level of the concrete.

2. The fatigue properties of lightweight aggregate concrete are not significantly different from the fatigue properties of normal weight concrete.

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Propiedades de fatiga del concreto liviano de agregado

Se realizaron ensayos de fatiga sobre dos concretos livianos de agregado diferentes, uno proporcional para alta resistencia y el otro para baja resistencia. Espécimenes de aproximadamente la misma edad fueron ensayos a niveles de esfuerzo de 40, 50, 60, 70, y 80 por ciento de la resistencia final de la compresión estática de las mezclas respectivas. Dentro de los límites de las investigaciones, el comportamiento de fatiga del concreto liviano de alta resistencia fué semejante al del concreto liviano de baja resistencia. Además, el comportamiento de fatiga del concreto liviano de agregado parece ser semejante al que se encuentra para un concreto de peso normal de un estudio anterior.

Les caractéristiques de fatigue du béton léger à agrégat

Des essais de fatigue ont été conduits sur deux types de béton léger à agrégat, l'un proportionné pour la haute résistance et l'autre pour la résistance faible. Des spécimens, ayant à peu près le même âge ont été essayés aux niveaux de 40, 50, 60, 70, et 80 pourcent de l'effort compressif statique définitif des mélanges réspectifs. Dans le cadre de la recherche, le comportement sous fatigue du béton léger à haute résistance était analogue à celui du béton léger à faible résistance. En outre, le comportement sous fatigue du béton léger à agrégat paraît être analogue à celui trouvé pour un béton de poids normal au cours d'une étude antérieure.

Ermuedungseigenschaften von Leichtmischbeton

Ermuedungsversuche wurden an zwei verschiedenen Arten von Leichtmischbeton durchgefuehrt, eine Mischung fuer hohe Festikgeit und eine fuer niedrige Festigkeit. Muster von ungefaehr gleichem Alter wurden unter Beanspruchungen von 40, 50, 60, 70 und 80 Prozent der absoluten statischen Druckfestigkeit der

jeweiligen Mischungen geprueft. Innerhalb der Grenzen der Untersuchung war das Ermuedungsverhalten der Leichtmischbetone mit hoher Festigkeit aehnlich den Leichtmischbetonen mit niederer Festigkeit. Ausserdem scheint das Ermuedungsverhalten von Leichtmischbeton aehnlich dem Verhalten, das fuer Normalbeton in einer frueheren Untersuchung gefunden wurde, zu sein.

Title No. 58-7

Based partly on work done in one laboratory as a part of a cooperative investigation sponsored by ACI Committees 213 and 716; the total investigation was conducted in four laboratories. Because this paper includes one more test method than the work of the committees, it is being published separately. The companion paper, covering the work of four laboratories, will appear in a later issue.

Comparison of Four Different Methods of Determining Drying Shrinkage of Concrete Masonry Units

By JAMES O. BRYSON and DAVID WATSTEIN

Four different procedures for determining the drying shrinkage of concrete masonry units were compared to determine their suitability as possible standard test methods. The test procedures differed mainly by the conditions under which the specimens were dried and were designated RT-50 (73 F and 50 percent relative humidity), RT-30 (73 F and 30 percent relative humidity), Modified British (122 F and 17 percent relative humidity), and Rapid (220-235 F). In addition to varying the drying conditions, the size and shape of test specimens were also varied. The drying-shrinkage test was performed using all four procedures on both autoclaved and low-pressure steam cured block of sand and gravel, cinders, expanded slag, expanded shale, and pumice aggregates.

■ THE DIMENSIONAL CHANGES of concrete masonry units are a problem of great concern to those conneeted with building construction. These units, in addition to varying in dimensions with temperature, undergo shrinkage due to drying. The degree to which a concrete masonry unit will change in volume on drying is known to depend primarily on the type of aggregate used in the concrete and the method of curing it received. Masonry walls constructed with concrete units that shrink excessively often develop cracks that are unsightly and sometimes damaging.

There is at present a renewed interest in the development of a rapid and reproducible test procedure to serve as a standard method for determining the potential drying shrinkage of concrete block in an acceptance test. This study was an attempt to evaluate the existing test procedures as well as an effort to develop an accelerated shrinkage test at room temperature.

SCOPE OF THE PROGRAM

Measurements were made of the shrinkage caused by four different procedures of drying of concrete masonry block made from five different types of aggregate and cured by two different methods. The drying procedures were designated as RT-50 (73 F and 50 percent relative humidity), RT-30 (73 F and 30 percent relative humidity), Modified British (122 F and 17 percent relative humidity), and Rapid (220 to 235 F) methods. The aggregate types were sand and gravel, cinders, expanded blast furnace slag, expanded shale, and pumice. In each case the block were supplied by a commercial source which normally produces them. All block were 8 x 8 x 16-in. units and their description is given below:

Block 1L—Expanded blast furnace slag aggregate with Type IS-A cement
Block 2L—Expanded shale aggregate with Type IS-A cement
Block 3L—Sand and gravel aggregates with Type I-A cement
Block 4L—Pumice aggregate with Type I cement
Block 5L—Cinder aggregate with Type III cement and fly ash
Block 6H—Expanded blast furnace slag aggregate with Type I-A cement and fly ash
Block 7H—Expanded shale aggregate with Type I-A cement and fly ash
Block 8H—Sand and gravel aggregates with Type I-A cement and silica flour
Block 9H—Pumice aggregate with Type I cement and silica flour
Block 10H—Cinder aggregate with Type I-A cement and fly ash

All block with the exception of Block 5L had two cells; Block 5L had three cells. Block 1L through 5L were low-pressure steam cured, while Block 6H through 10H were high-pressure steam cured.

The physical properties of the block are given in Table 1. The data were obtained in accordance with ASTM C 140-56, Sampling and Testing

TABLE 1 — PROPERTIES OF BLOCK

Block designation	Moisture content as received, percent of total absorption	Absorption		Unit weight, lb per cu ft	Compressive strength, psi		Dynamic Young's modulus,† 10^6 psi
		lb per cu ft	percent		Gross area	Net area	
1L	20.6	14.4	13.7	104.6	980	1970	1.61
2L	27.8	14.7	18.3	80.0	980	2020	1.35
3L	11.7	7.7	5.6	136.0	1520	2800	3.62
4L	45.1	20.5	32.2	63.6	760	1410	0.76
5L	19.3	13.4	15.2	87.86	890*	1570	1.04
6H	5.4	15.8	15.2	103.7	1020	2060	2.24
7H	14.2	13.1	16.1	81.1	1790	3340	1.74
8H	2.3	8.6	6.4	133.1	1840	3410	3.12
9H	41.7	18.4	28.6	64.4	1010	1800	0.92
10H	26.6	14.5	15.6	93.0	1500*	2840	1.58

*One block tested. The other values are the average of two blocks.
†Duplicate prisms 1¼ x 1¼ x 8 in. were used. They were cut from the face shells with the long dimension taken in the direction of the longitudinal axis of the block.
Note: The methods of ASTM C 140-56 and C 215-58T were used for the applicable tests.

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Concrete Masonry Units, and ASTM C 215-58T, Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens.

Care was taken to have the block delivered in time to be placed under test when they were 28 days old. However, due to some inadvertent delays, some block were placed under test about 2 to 3 weeks late.

Various types of specimens representing these block were placed under test using the four methods of drying.

In addition to the shrinkage tests made on the units which were 28 days old, a second group of specimens were placed under test by the RT-30 Method after they were aged indoors for about 4 months.

TEST PROCEDURES AND TEST SPECIMENS

All block were placed in a room controlled at a temperature of 73 F and 50 percent relative humidity for a minimum of 7 days prior to processing. At the end of this period the necessary cutting was done using a 22 in. diamond saw with a water stream applied to the cutting edge. Gage plugs made from brass machine screws were set in drilled holes in the specimens with neat cement paste.

All specimens of the same aggregate-cure combination were placed under test at the same time. The specimens were submerged in water at 73 ± 3 F for 48 hr prior to the test cycle, and the initial measurements of length were made immediately after the soaking period. A Whitemore strain gage equipped with a dial reading to 0.0001 in. was used for measuring length changes over a 10 in. gage length on all specimens.

The procedure and test specimens used in this study are described below.

RT-50 Method

The specimens were dried in a room where the temperature was controlled at 73 ± 3 F and the relative humidity was controlled at 50 ± 5 percent. The specimens were considered to be at equilibrium with the drying conditions when their average change in length was not greater than 0.002 percent over 14 days.

Modified British Method[1]

Following the initial measurements, which in all cases were scheduled to fall on a Friday, the specimens for this method were placed in a room controlled at 73 ± 3 F and 50 ± 5 percent relative humidity over a weekend, after which they were placed in an oven in which the temperature was controlled at 122 ± 2 F and the humidity controlled by a saturated solution of calcium chloride. These conditions have been reported as providing a relative humidity of about 17 percent.[2] The initial period of oven storage was 5 days. This was followed by

cooling over the weekend in air-tight drums. Length measurements were made at the end of the cooling period. Subsequent drying periods of 48 hr at 122 F followed by an overnight cooling period during the week, and a cooling period over the weekend, were continued until equilibrium was reached. The specimens were considered to be at equilibrium when the average total length change over three 48 hr drying periods did not exceed 0.002 percent.

Rapid Method

The specimens were dried in an oven in which the temperature was controlled at 220 to 235 F for 48 hr, after which they were placed in air-tight cooling drums located in a room controlled at 73 ± 3 F. The measurements were made on the specimens after they had cooled to the temperature of the room. Additional drying periods of 24 hr followed by cooling in the drums were continued until equilibrium was reached. The length of each cooling period was 24 hr. Equilibrium was considered to have been attained when the average change in length during oven drying for 24 hr did not exceed 0.002 percent.

RT-30 Method

The specimens were dried in a room in which the temperature was controlled at 73 ± 3 F and the relative humidity was controlled at 30 ± 2 percent. Equilibrium was considered to have been attained when the average change in length did not exceed 0.002 percent over 14 days.

Procedure used in determining the equilibrium shrinkage values

The values of equilibrium shrinkage for all specimens were obtained from shrinkage-time curves drawn through experimental points. It is noted that in each of the four test methods described previously the equilibrium shrinkage is defined as the value corresponding to a certain prescribed rate of shrinkage. The graphical method of obtaining the equilibrium shrinkage values from the curves is shown in Fig. 1. The secant procedure, also illustrated in this figure, is discussed later.

The dotted line having a slope corresponding to the limiting value of shrinkage rate was fitted to the experimental curve in such a manner that the points of intersection of this line intercepted the proper time interval; these time intervals corresponded to the periods cited under the definitions of equilibrium drying shrinkages for the four test procedures used in this study. Thus, for the RT-30 and RT-50 methods, this time interval between Points 1 and 2 was 14 days and the corresponding increment of shrinkage was 0.002 percent. For the Modified British Method, the time interval 1-2 was 6 days and the increment in shrinkage was 0.002 percent. For the Rapid Method, the time interval 1-2 was 1 day and the shrinkage increment was 0.002 percent. The value of equilibrium shrinkage

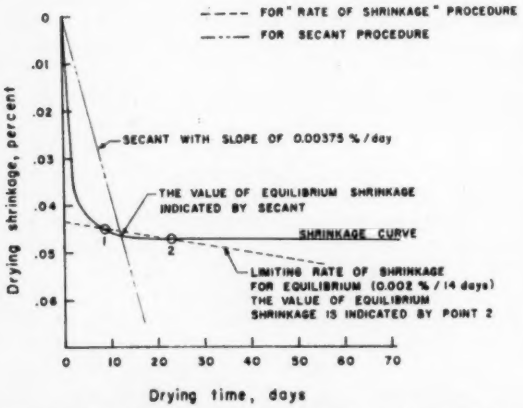

Fig. 1—Illustration of how the value of equilibrium shrinkage was determined by both the rate of shrinkage and secant procedures

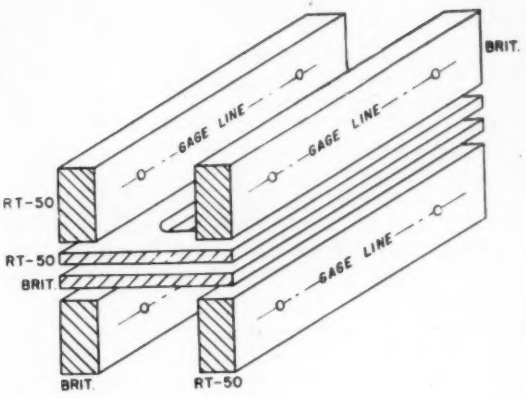

Fig. 2—Half shells and thin laminas shown in the positions from which they were cut from block

determined for all four methods in the manner indicated in Fig. 1, was the shrinkage corresponding to Point 2.

Test specimens

The test specimens used in this study are illustrated in Fig. 2 and 3. Whole-block specimens were tested by each of the four methods for each of the aggregate-cure combinations. In addition to whole-block specimens, representative specimens were cut and prepared for length measurements. Two blocks of each aggregate-cure combination were prepared as shown in Fig. 2 for the RT-50 and Modified British methods. This figure shows the half-shell and lamina specimens in the positions from which they were removed from the block. It can be seen that four half-shell and two lamina specimens were obtained from a block. One lamina and two diagonally opposite half shells were tested by the RT-50 Method and the remaining lamina and two half-shell specimens were tested by the Modified British Method. Accordingly, the specimens that represented an aggregate-cure combination in the RT-50 and Modified British tests were two whole block, two laminas, and four half shells for each method.

The specimens that were used in the tests of block of each aggregate-cure combination by the RT-30 Method were one whole block, one remnant block, and two laminas. Fig. 3 shows the remnant and lamina specimens outlined on a whole block.

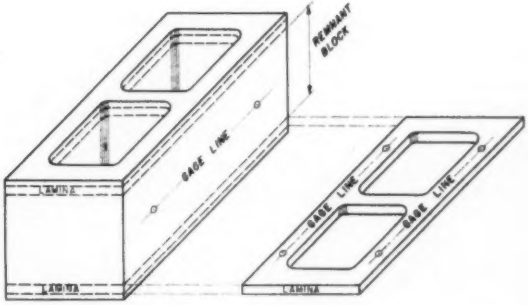

Fig. 3—Relationship of the thin laminas and remnant block to the whole block for the RT-30 test

Test specimens for the Rapid Method were whole block, tested in duplicate.

The location of gage plugs which received the points of the 10-in. gage are shown in Fig. 3 for the whole block, remnant block, and thin lamina sections. Gage plugs were installed on both faces of whole block and remnant block. The location of these gage points on the half shells is indicated in Fig. 2. It is noted that gage lines were established both on the inner and outer faces of the half shells.

The thin lamina specimens respond rapidly to changes in environment during fabrication. Considerable care should be taken, therefore, to follow in detail a standardized procedure such as described hereafter to avoid the irreversible length changes which occur in sections permitted to dry partially during their preparation.

The lamina specimens were cut using a 22 in. diamond cutting wheel provided with depth and cross-feed controls. Diamond cutting wheels, of course, must be used with a liberal amount of water applied as a jet at the cutting edge. The slices were ½ in. thick. The cut through an 8 x 8 x 16-in. unit is made in two passes of the diamond cutting wheel.

After the first cut was made, holes for the gage plugs were drilled sufficiently deep to assure that the lamina specimen was perforated after it was removed by completion of the second cut. The holes for the gage reference plugs in the laminas as well as in the remnants and whole blocks were drilled with a ¼-in. (#12) carbide tipped drill. The lamina specimens were submerged in water immediately after cutting. The laminas were removed from the water one at a time for installation of the plugs. As the gage plugs were installed, the specimens were immediately covered with damp burlap and kept moist for 24 hr.

The reference holes in gage plugs were then drilled using a high speed 1.2-mm drill. The specimens were removed from under the burlap one at a time and were placed in water immediately after the holes were drilled, reamed, and checked with a 10-in. gage for reproducibility of readings. The soaking period was reckoned from the time the specimens were returned to the water following processing of the gage plugs.

TABLE 2 — EQUILIBRIUM DRYING SHRINKAGE VALUES IN PERCENT AND

Method	Specimen	Block							
		1L		2L		3L		4L	
RT-50	Whole block	0.0459	0.0009	0.0545	0.0048	0.0233	0.0033	0.0776	0.0031
	Half shell	0.0552	0.0027	0.0629	0.0034	0.0261	0.0055	0.0792	0.0044
	Lamina	0.0645	0.0019	0.0654	0.0038	0.0336†		0.0831	0.0032
Modified British	Whole block	0.0451	0.0026	0.0549	0.0058	0.0299	0.0018	0.0802§	0.0084
	Half shell	0.0479‡	0.0023	0.0605	0.0010	0.0330	0.0040	0.0790§	0.0019
	Lamina	0.0575	0.0008	0.0693	0.0035	0.0327†		0.0785	0.0027
Rapid	Whole block	0.0561	0.0018	0.0669	0.0045	0.0450	0.0000	0.2055	0.0010
RT-30	Whole block	0.0413		0.0523		0.0280		0.0804	
	Remnant	0.0470		0.0500		0.0283		0.0816	
	Lamina	0.0514	0.0007	0.0519	0.0005	0.0279	0.0002	0.0832	0.0006

*The value in the right-hand column under each block designation is the difference between maximum and minimum values of shrinkage at equilibrium when two or more specimens were tested. Where no value is given, only one specimen was tested.

†One specimen

‡Three specimens

§Criterion for equilibrium shrinkage was not met in 23 days of drying. The value reported is the maximum observed value of shrinkage.

**Maximum values

RESULTS AND DISCUSSION

The equilibrium shrinkage values and the spread between the maximum and minimum measurements at equilibrium for the different groups of specimens tested by each of the four methods are given in Table 2. These shrinkage values are the averages for the respective types of specimens and, as previously stated, were obtained from experimental curves of the shrinkage-time data. The values of shrinkage obtained from these curves correspond to the conditions of equilibrium previously defined.

The shrinkage-time curves for the specimens tested by the RT-50, RT-30, and Modified British methods are presented in Fig. 4 through 8. Curves for the Rapid Method tests are not presented.

Effect of specimen size on value of equilibrium shrinkage

It is immediately apparent from the data shown in Fig. 4 through 8 that for autoclaved block the size of the specimen has no important effect on the equilibrium value of shrinkage for a given test method. For the autoclaved block the shrinkage-time curves obtained with the three types of specimens fell within a narrow band and in a random order for all three test methods shown. However, the average values of shrinkage determined by the three methods showed measurable differences. It was noted that the Modified British values were somewhat higher than either the RT-50 or RT-30 values. When the data obtained on all types of specimens from autoclaved block are averaged, it is found that the ratios of Modified British to RT-50 values ranged

SPREAD* BETWEEN MAXIMUM AND MINIMUM MEASUREMENTS

designation												
5L		6H		7H		8H		9H		10H		
0.0428	0.0018	0.0158	0.0013	0.0221	0.0015	0.0165	0.0004	0.0344	0.0027	0.0203	0.0012	
0.0466	0.0026	0.0149	0.0007	0.0243	0.0006	0.0182	0.0010	0.0372	0.0029	0.0185	0.0016	
0.0565	0.0069	0.0151	0.0011	0.0256	0.0022	0.0174	0.0035	0.0357	0.0059	0.0191	0.0011	
0.0392	0.0052	0.0213	0.0004	0.0229	0.0033	0.0200	0.0020	0.0400	0.0000	0.0240	0.0004	
0.0419	0.0000	0.0187	0.0009	0.0243	0.0032	0.0209	0.0007	0.0381	0.0000	0.0220	0.0004	
0.0500	0.0084	0.0189	0.0019	0.0276	0.0014	0.0210	0.0000	0.0349	0.0018	0.0230	0.0000	
0.0547	0.0034	0.0370	0.0021	0.0540	0.0000	0.0358	0.0005	0.1315	0.0030	0.0437	0.0034	
0.0438		0.0107		0.0222		0.0178		0.0394**		0.0190		
0.0449		0.0127		0.0210		0.0159		0.0412**		0.0199		
0.0470	0.0068	0.0120	0.0008	0.0212	0.0012	0.0200	0.0033	0.0331	0.0024	0.0200	0.0007	

Note: In the RT-50 and the Modified British methods, whole block and laminas were tested in duplicate and half shell specimens in quadruplicate. In the Rapid Method, the whole block specimens were tested in duplicate. In the RT-30 Method, only one whole block and one remnant block of each aggregate-cure combination were tested, while the laminas were tested in duplicate.

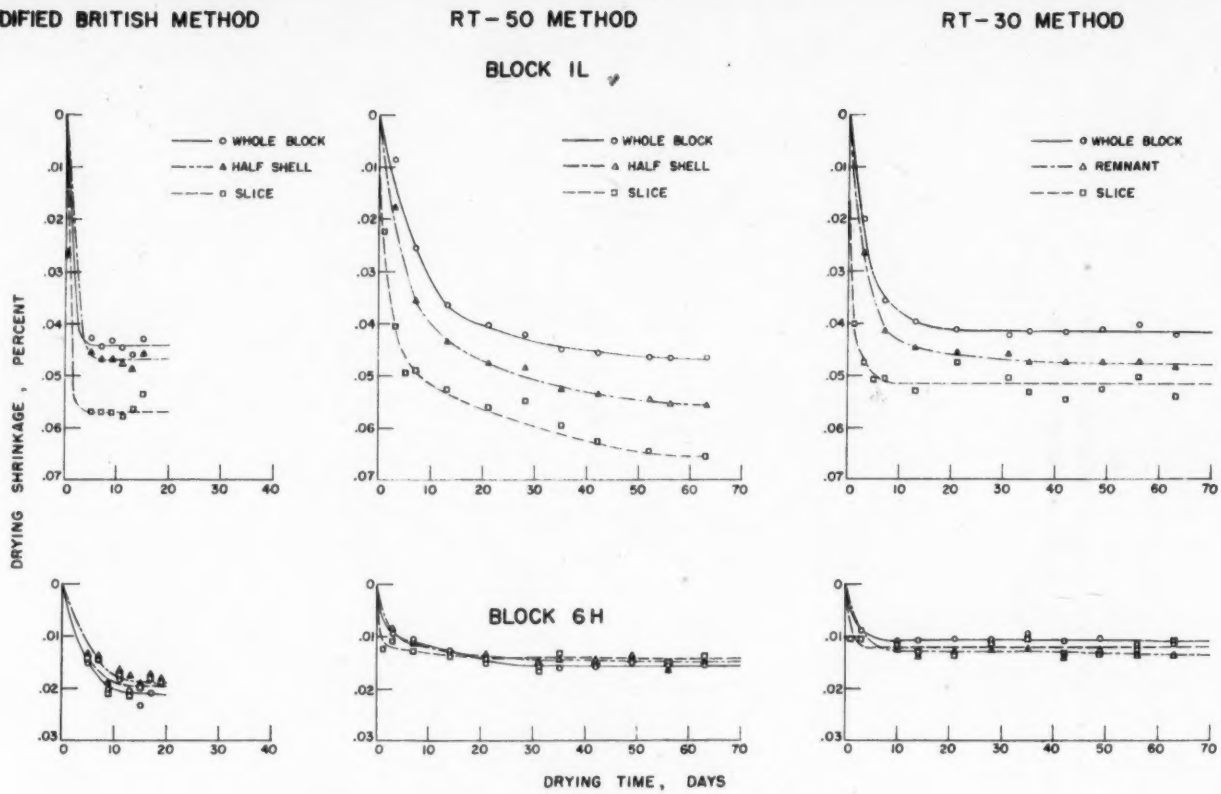

Fig. 4—Shrinkage versus time curves, Block 1L and 6H

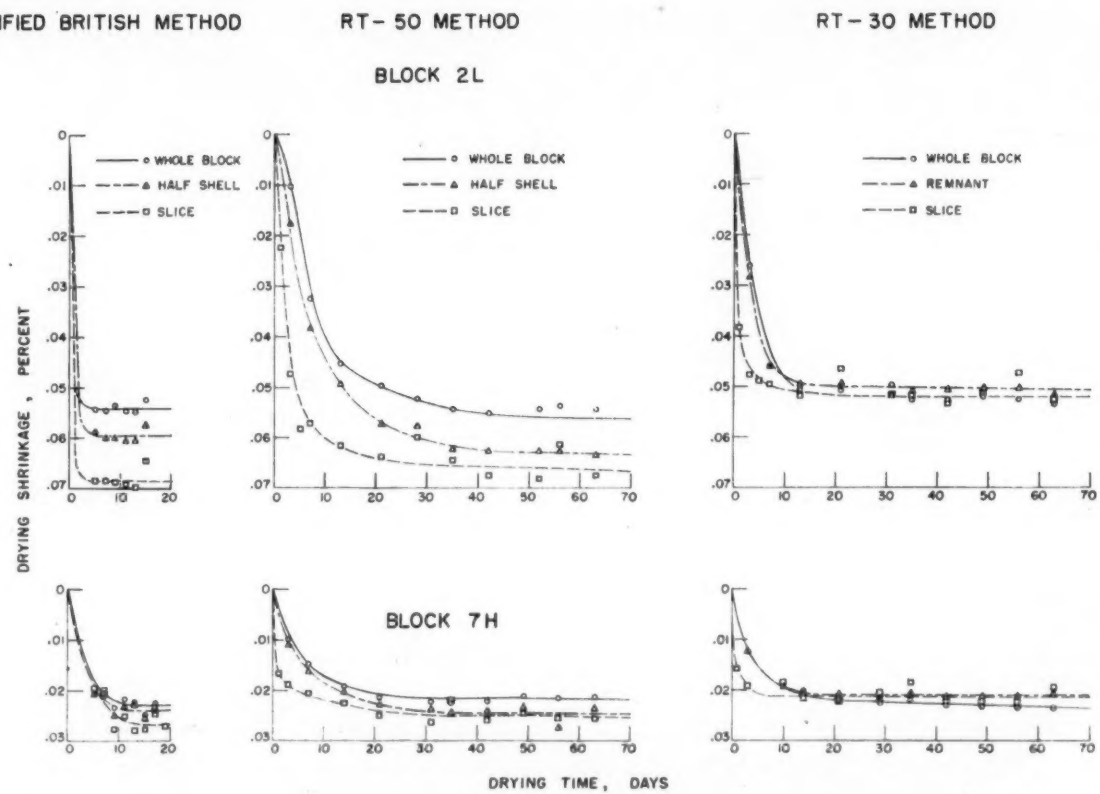

Fig. 5—Shrinkage versus time curves, Block 2L and 7H

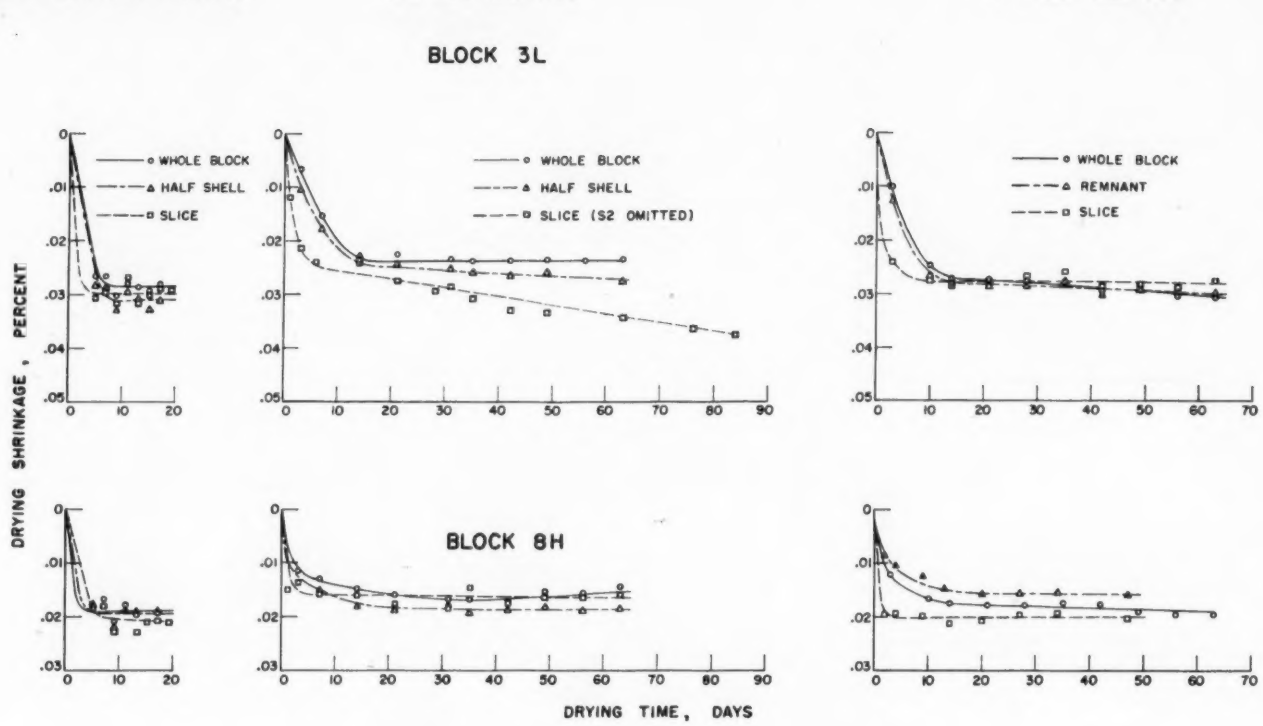

Fig. 6—Shrinkage versus time curves, Block 3L and 8H

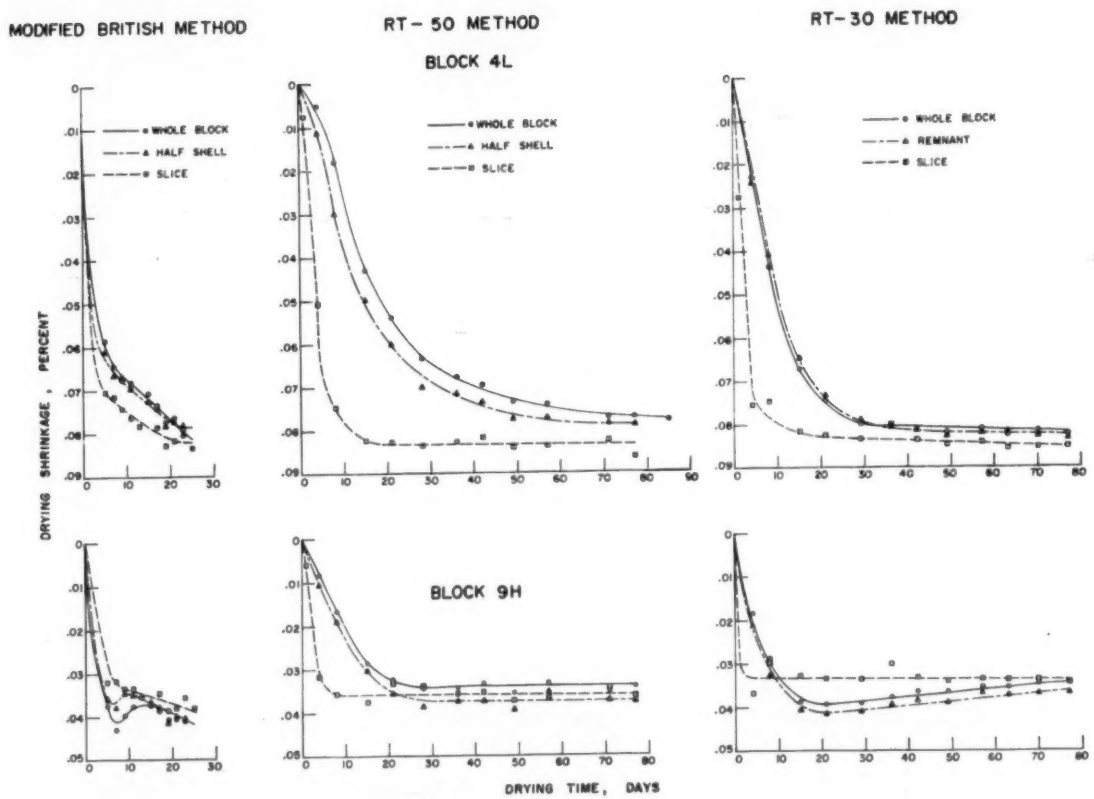

Fig. 7—Shrinkage versus time curves, Block 4L and 9H

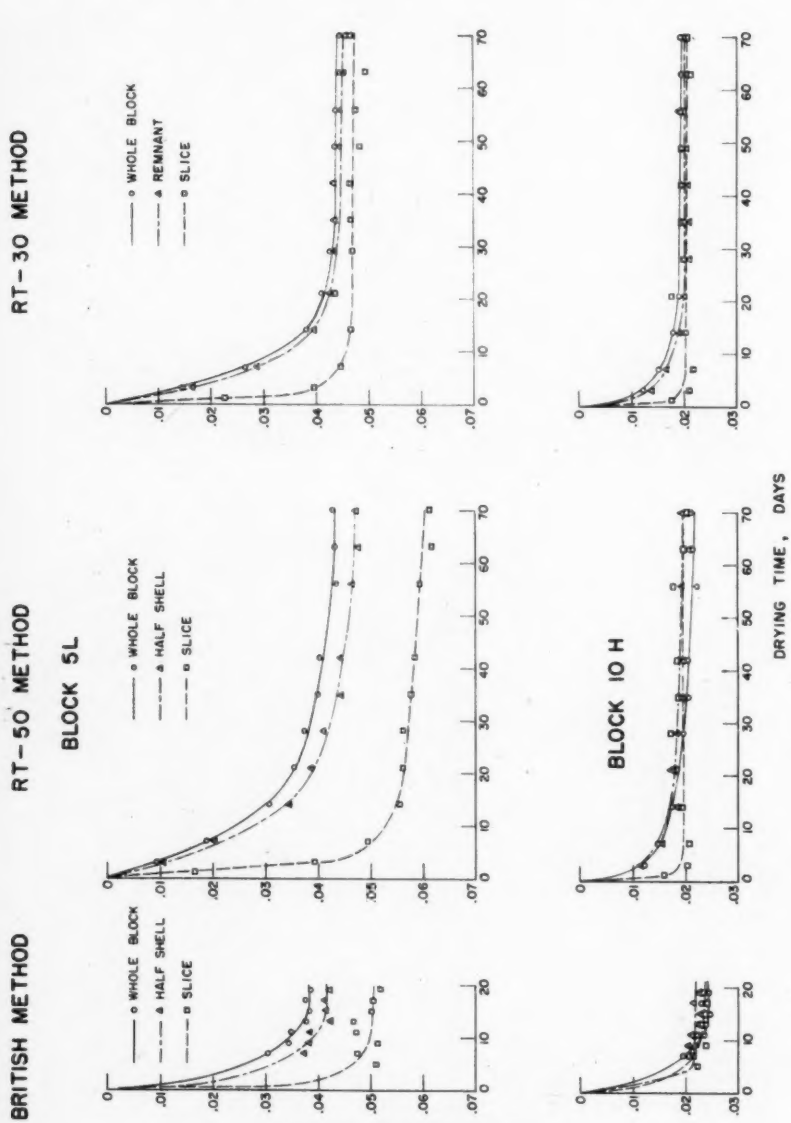

Fig. 8—Shrinkage versus time curves, Block 5L and 10H

from 1.04 to 1.28, while the ratios of RT-30 to RT-50 values ranged from 0.77 to 1.06.

For low-pressure steam cured block, however, the size of the specimens had a significant effect on the shrinkage obtained by the Modified British and RT-50 methods. Without exception, the thin laminas showed the greatest shrinkage, the half-shell specimens showed intermediate, and the whole-block specimens showed the least.

In general, the thin laminas also gave somewhat higher shrinkage values when tested by the RT-30 method than either the remnant or whole-block specimens. However, the differences observed between the thin laminas and the parent remnant block in the RT-30 series were considerably smaller than the corresponding differences for the RT-50 series. Thus, as can be found from Table 2, the differences between equilibrium shrinkage values for the laminas and half-shell specimens ranged up to 0.01 percent in the RT-50 series, and to a maximum of 0.004 percent in the RT-30 series. It is noted that in making this comparison the half-shell specimens and the remnant-block specimens are considered as comparable test specimens, although the latter are four times the size.

Variation of the rate of shrinkage in specimens approaching equilibrium

An important distinction between the results obtained by the RT-30 and the RT-50 methods is evidenced by a comparison of the slopes of the time-shrinkage curves after the first 28 days of drying. The values of shrinkage rates, expressed in percent shrinkage per day, for all type specimens tested by the two methods are given in Table 3. It is seen that, in general, the low-pressure steam cured block dried at 30 percent relative humidity (the RT-30 Method) exhibited a measurably lower rate of shrinkage as they approached a state of equilibrium as defined for the methods involving room temperature than did the specimens dried by the RT-50 Method. The shrinkage rate values for low-pressure steam cured block 1L, 4L, and 5L may be singled out in this respect.

Because of the difference in the nature of the drying process and the test procedure in general, similar comparisons of the results from tests by the Modified British Method are not given. However, it is noted that for low-pressure steam cured specimens from Block 4L tested by this method the criterion for equilibrium shrinkage was not met by the end of the test after 23 days drying in the oven.

The foregoing remarks concerning the differences in the rates of shrinkage obtained by the RT-50 and RT-30 methods as the block approached equilibrium are not applicable to the autoclaved block. It is apparent from an examination of these data that in general the slopes of the shrinkage-time curves nearing the ends of the drying cycles were nearly the same for both the RT-50 and RT-30 methods. Only in the

TABLE 3 — SHRINKAGE RATES FOR THE RT-50 AND RT-30 METHODS OCCURRING AFTER THE FIRST 28 DAYS OF DRYING

Method	Specimen	Shrinkage rate, 10^{-4} percent per day									
		Block designation									
		1L	2L	3L	4L	5L	6H	7H	8H	9H	10H
RT-50	Whole block	1.20	0.60	0.00	2.30	1.30	0.00	0.00	0.00	0.00	0.50
	Half shell	1.90	1.00	0.60	2.00	1.40	0.00	0.10	0.00	0.00	0.20
	Slice	2.20	0.40	1.60	0.00	0.80	0.00	0.10	0.00	0.00	0.00
RT-30	Whole block	0.00	0.30	0.90	0.40	0.30	0.00	0.30	0.50	0.00	0.00
	Remnant block	0.40	0.20	0.40	0.30	0.50	0.20	0.00	0.00	0.00	0.00
	Slice	0.00	0.00	0.00	0.40	0.00	0.00	0.00	0.00	0.10	0.00

case of Block 9H was there a significant departure from this observation; the whole-block and remnant-block specimens in the RT-30 series showed a reversal in the shrinkage movement. It is significant that the laminas cut from autoclaved Block 9H did not exhibit such a reversal in the shrinkage movement, suggesting that the presence of significant amounts of moisture in the slower drying larger specimens was essential to the unknown mechanism responsible for this reversal of shrinkage.

Comparison of equilibrium shrinkage values from the four methods

In view of the inconsistency of the effect of specimen size on shrinkage values, a comparison of the results for the different methods was made using only the values obtained from whole block. Table 4 gives the equilibrium shrinkage ratios for whole-block specimens tested by the Modified British, Rapid, and RT-30 methods using the results from the RT-50 tests as a basis of comparison. The RT-50 Method was selected as a basis of comparison since, of all four methods evaluated, it represents most closely the actual service conditions to which concrete masonry is exposed. The shrinkage ratios show that the Modified British and RT-30 whole-block specimens shrink about the same amount as the whole-block specimens tested by the RT-50 Method. The specimens from the Modified British tests had shrinkage ratios for the different aggregate-cure combinations ranging from a low of 0.92 for low-pressure units of Block 5L to a high of 1.35 for autoclaved units of Block 6H, while the ratios for the RT-30 specimens ranged from 0.68 for autoclaved units, Block 6H, to 1.20 for low pressure units, Block 3L. The ratios for the specimens tested by the Rapid Method show that drying shrinkage measured by this method bore no consistent relationship to the values obtained by the RT-50 Method. For the low-pressure

TABLE 4 — SHRINKAGE RATIOS FOR WHOLE BLOCK WITH THE RT-50 SERIES AS A BASE

Test method being compared	Block designation									
	1L	2L	3L	4L	5L	6H	7H	8H	9H	10H
Modified British	0.98	1.01	1.28	1.03	0.92	1.35	1.04	1.21	1.16	1.18
Rapid	1.22	1.23	1.93	2.65	1.28	2.34	2.44	2.17	3.82	2.15
RT-30	0.90	0.96	1.20	1.04	1.02	0.68	1.00	1.08	1.15	0.94

TABLE 5 — SHRINKAGE OBSERVED IN WHOLE-BLOCK SPECIMENS BEYOND THE EQUILIBRIUM VALUES AS DEFINED BY THE CRITERION FOR THE RT-50 SERIES

Block designation	Drying period at equilibrium, days	Shrinkage at equilibrium, percent	Maximum drying period, days	Maximum observed shrinkage, percent	Increase in shrinkage beyond equilibrium, percent
1L	46	0.0459	201	0.0580	26
3L	27	0.0233	178	0.0308	32
4L	77	0.0776	153	0.0846	9
5L	54	0.0428	138	0.0498	16

steam cured block the ratios of Rapid to RT-50 values ranged from 1.22 to 2.65; for autoclaved block, these ratios ranged from 2.15 to 3.82. It is obvious that if the RT-50 values for whole-block specimens are to be considered as being most nearly representative of the performance of the block under actual service conditions, then the values obtained by the Rapid Method cannot be relied on for predicting the performance of masonry in service.

Shrinkage in specimens allowed to dry for extended periods beyond equilibrium in the RT-50 test

It is generally assumed that the equilibrium shrinkage values found with the RT-50 Method represent nearly the maximum values of shrinkage which the block develop at that temperature and humidity. To verify this hypothesis, several whole-block specimens, all low-pressure steam cured, were permitted to dry for a prolonged period following the attainment of equilibrium shrinkage values. The results of these few tests are given in Table 5. It can be seen that the increases in the drying shrinkage during the additional drying periods ranged from 9 percent for Block 4L to 32 percent for Block 3L. It appears that shrinkage studies of concrete masonry units, particularly the low-pressure steam cured variety, should be carried out over considerably longer periods than were used in this study to arrive at the ultimate shrinkage values.

The tendency for low-pressure cured block, tested by the RT-50 Method, to continue to shrink for long periods may possibly be explained

TABLE 6 — EQUILIBRIUM* SHRINKAGE VALUES AND DRYING PERIODS NEEDED TO ATTAIN EQUILIBRIUM FOR LAMINAS DRIED BY THE RT-30 METHOD

Block designation	Equilibrium shrinkage values, percent		Drying period required to attain equilibrium, days	
	Secant method	Rate-of-shrinkage method	Secant method	Rate-of-shrinkage method
1L	0.051	0.051	14	19
2L	0.052	0.052	14	22
3L	0.027	0.028	7	19
4L	0.083	0.083	22	27
5L	0.046	0.047	12	23
6H	0.012	0.012	4	16
7H	0.021	0.021	6	17
8H	0.020	0.020	6	15
9H	0.033	0.033	9	15
10H	0.020	0.020	6	16

*In the secant method, the equilibrium shrinkage value is defined as the point of intersection of the shrinkage-time curve with a secant drawn from the origin and having a slope of 0.00375 percent per day. The rate-of-shrinkage method is based on the criterion that equilibrium is attained when the average change in length does not exceed 0.002 percent over 14 days.

by the fact that in addition to the drying shrinkage of concrete, carbonation shrinkage also develops under favorable conditions. Results reported by other investigators[3,4] indicate that the conditions of drying used in the RT-50 Method are conducive to carbonation shrinkage. Verbeck[4] reported that carbonation shrinkage was appreciable at 50 percent relative humidity, but was quite small at 30 percent relative humidity. This fact would seem to offer an explanation of the significant differerties of block, a second group of specimens was tested by the RT-30 and RT-50 series. The relatively good concordance of the shrinkage values obtained with the laminas, the parent remnant block, and the whole-block specimens in the RT-30 series is probably a direct consequence of low carbonation shrinkage at a relative humidity of 30 percent.

Effect of prolonged storage on the shrinkage of block

To determine the effect of prolonged storage on the shrinkage properties of block, a second group of specimens was tested by the RT-30 Method. The specimens were obtained from block that were left over from the original supply. The block had been stored for 4 months during the summer in a room where there was no temperature or humidity control. The shrinkage-time curves for this second group are presented in Fig. 9. It can be seen that the shrinkage curves for the second group of autoclaved specimens were practically identical with those obtained from the first group. However, the second group of low-pressure cured specimens had shrinkage equilibrium values that were reduced from 25 to 50 percent compared with the shrinkage of similar specimens from the first test. The results from this second test indicate that the

shrinkage of low-pressure cured block is significantly reduced after several months of indoor storage, whereas the shrinkage of autoclaved block is apparently unaffected by such storage.

Length of drying periods needed to reach equilibrium

In addition to judging a given method by the value of shrinkage it yields, the suitability of a drying shrinkage test procedure is also judged by the length of time required to attain equilibrium. The ages at equilibrium for the Modified British, the RT-50, the RT-30, and the Rapid methods are shown in Fig. 10 and 11.

In the RT-50 Method the period of drying of whole-block specimens to equilibrium ranged from 27 to 77 days for low-pressure cured block, and from 26 to 34 days for autoclaved block. The use of laminas in

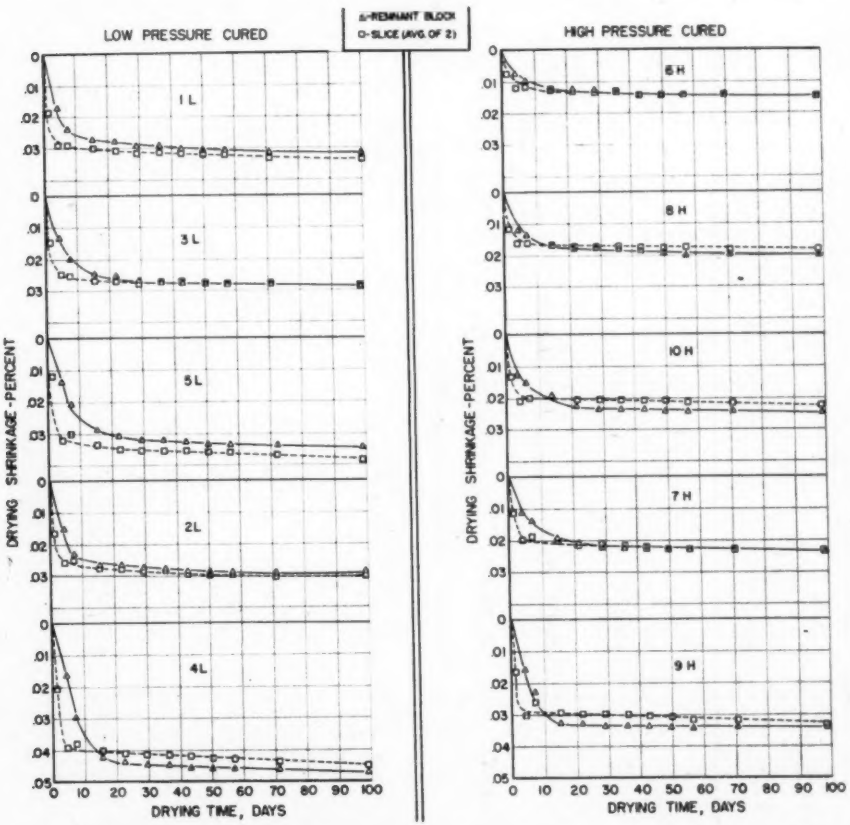

Fig. 9—Shrinkage versus time curves for the second group of specimens tested by the RT-30 Method; (The second group of specimens were aged indoors for 4 months before testing)

the RT-50 series did not produce a uniform reduction in the time required to attain equilibrium. In 8 out of 10 cases, the use of laminas resulted in a reduction in the drying time. In the other two cases the period of drying to equilibrium was longer for the laminas than for the other specimens.

In the Modified British series, the period of drying of whole-block specimens to equilibrium ranged from 11 to 23 days for low-pressure cured block and from 11 to 15 days for autoclaved block. The use of half shells and laminas did not appreciably affect the length of drying to equilibrium. Note that the time scale for this method given in Fig. 4 through 8 represents oven drying only. The Modified British Method generally requires a much shorter drying time for specimens to reach equilibrium than is required by the RT-50 Method. However, when the time required for cooling is added to the oven drying time, as many as 40 days may be required to reach equilibrium. This was the case for Block 4L specimens (it is noted that for every 4 days of oven drying there were approximately 3 days of cooling).

In the Rapid Method equilibrium was established in a much shorter time than was required by any of the other methods. In no case was as much as a week of drying needed to bring about equilibrium. The average number of drying days needed to meet the equilibrium require-

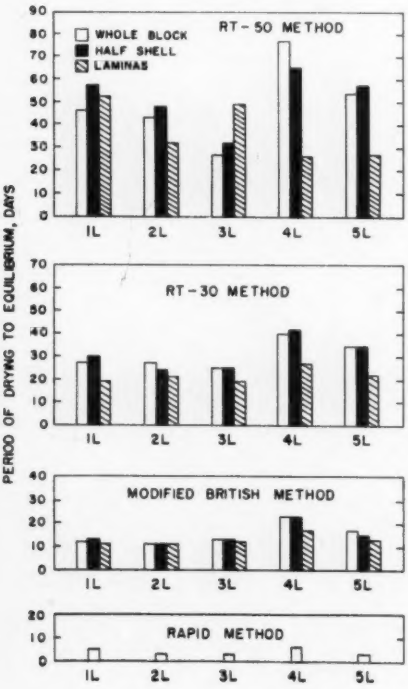

Fig. 10—Period of drying to equilibrium, low pressure steam cured blocks

Note:

1. The period of drying to equilibrium for the RT-50 and RT-30 specimens is defined as the drying time at the end of the first 14 days during which the shrinkage change was 0.002 percent.

2. The period of drying to equilibrium for the Modified British specimens is defined as the drying time at the end of the first 6 days during which the shrinkage change was 0.002 percent.

3. The period of drying to equilibrium for the Rapid specimens is defined as the drying time at the end of the first 24 hr during which the shrinkage change was 0.002 percent.

ment was 4, and in only one case (low-pressure cured Block 4L) were as many as 6 days needed.

In the RT-30 series, the time required for whole-block and remnant block specimens to attain equilibrium ranged from 24 to 44 days for low-pressure cured block and 18 to 26 days for autoclaved block. The use of laminas reduced the time required from 19 to 27 days for low-pressure cured block and from 15 to 17 days for autoclaved block. It is noted that the criterion adopted for judging the attainment of equilibrium in the RT-30 series calls for 14 days of observations. Since both the low-pressure steam cured block and the autoclaved block exhibited a significantly lower rate of shrinkage as they approached equilibrium at 30 percent relative humidity than at 50 percent relative humidity, it was apparent that the RT-30 test could be accelerated by redefining equilibrium shrinkage. The equilibrium shrinkage for lamina specimens tested by the RT-30 Method was therefore redefined as the value corresponding to the point of intersection of the shrinkage-time curve with a secant drawn from the origin and having a slope of 0.00375 percent per day. Whole-block and remnant-block specimens were not included since their purpose in this test was to serve as controls. Fig. 1 shows an illustration of both the rate of shrinkage and secant procedures used for determining equilibrium shrinkage. Note that in determining the equilibrium shrinkage in accordance with the "rate of shrinkage" criterion, a line having a slope of 0.002 percent per 14 days is positioned on the shrinkage curve so that the line intersects the curve at two points 14 days apart. The second point on the curve indicates the value of equilibrium shrinkage. The comparative values of "equilibrium" shrinkage and the time required to attain these values are given in Table 6. From this table it is seen that the values of equilibrium shrinkage as determined by the two procedures are practically the same, but in all cases the drying periods needed to determine these values of shrinkage are considerably shorter for the secant procedure. For the low-pressure steam cured specimens the drying times needed to deter-

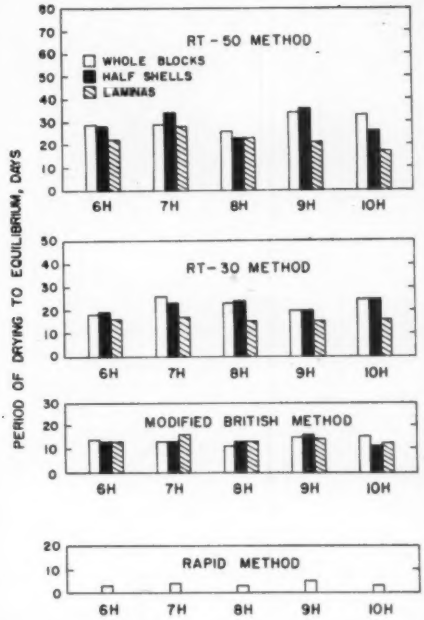

Fig. 11—Period of drying to equilibrium, high pressure steam cured blocks; (See Notes 1 through 3, Fig. 10)

mine the equilibrium shrinkages were reduced by from 5 to 12 days (drying ranged from 7 to 22 days) with the secant procedure and by from 6 to 12 days for autoclaved specimens (drying ranged from 4 to 9 days).

SUMMARY

1. The ratios of Modified British to RT-50 shrinkage values ranged from 0.92 to 1.35, while for RT-30 series these ratios ranged from 0.68 to 1.20.

2. The shrinkages obtained by the Rapid Method bore no consistent relationship to the values obtained by the RT-50 Method. For the low-pressure steam cured block the ratios of the Rapid to RT-50 values ranged from 1.22 to 2.65; for the autoclaved block these ratios ranged from 2.15 to 3.82.

3. The size of the specimen had no important effect on the equilibrium values of shrinkage for autoclaved block tested by a given method. However, the average values of shrinkage for the three types of specimens of autoclaved block determined by each of the three methods in which the size of specimen was a variable, showed measurable differences. With all specimens taken into account, the ratios of Modified British to RT-50 values ranged from 1.04 to 1.28, while for the RT-30 Method these ratios ranged from 0.77 to 1.06.

4. For the low-pressure steam cured specimens tested by the RT-50 and Modified British methods, the laminas showed the greatest shrinkage, the half shells showed intermediate values and the whole block showed the least values of shrinkage. In general, the thin laminas also gave somewhat higher shrinkage by the RT-30 Method than did either the remnant or whole-block specimens, but the differences were considerably smaller than the corresponding differences for the RT-50 series.

5. The rate of shrinkage of low-pressure steam cured block as they approached equilibrium was markedly lower when dried by the RT-30 Method than for block dried by the RT-50 Method. However, for autoclaved block the rate of shrinkage as the block approached equilibrium was nearly the same for both the RT-30 and RT-50 Methods.

6. The use of laminas did not produce a uniformly significant reduction in the time required to attain equilibrium by the RT-50 Method. However, in the RT-30 Method, the use of laminas reduced the time to attain equilibrium significantly in all cases. Additional significant reductions in drying times were realized for the lamina specimens in the RT-30 test by defining equilibrium shrinkage as the value of shrinkage corresponding to the point of intersection of the shrinkage-time curve and a secant line having a slope of 0.00375 percent shrinkage per day.

7. Tests of a group of block by the RT-30 Method after prolonged storage indoors indicated that shrinkage of low-pressure cured block is significantly reduced, whereas shrinkage of autoclaved block is apparently unaffected by such storage.

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Discussion of this paper should reach ACI headquarters in triplicate by Nov. 1, 1961, for publication in Part 2, March 1962 JOURNAL.

Comparación de cuatro métodos diferentes para determinar la contracción por desecación de la unidades de mampostería de concreto

Cuatro procedimientos diferentes para la determinación de la contracción por desecación de las unidades de mampostería de concreto feuron comparados para determinar su conveniencia como métodos posibles de ensayos de norma. Los procedimientos de ensayos se diferenciaban principalmente en las condiciones bajo las cuales los especímenes fueron secados y fueron designados RT-50 (73 F y humedad relativa de 50 por ciento), RT-30 (73 F y humedad relativa de 30 por ciento), procedimiento inglés modificado (122 F y humedad relativa de 17 por ciento), y procedimiento rápido (220-235 F). Además de variar las condiciones de secamiento, el tamaño y la forma de los especímenes de ensayo también fueron variados. El ensayo de la contracción por desecación se realizó con el empleo de todos los cuatro procedimientos en bloques curados en autoclave y de vapor de baja presión, de arena y grava, cenizas, escorias dilatadas, esquisto dilatado, y agregados de piedra pómez.

Comparaison de quatre méthodes différentes pour détermination de la retraite pendant le séchage d'éléments de construction en béton

On fait la comparaison de quatre procédés différents pour déterminer la retraite pendant le séchage d'éléments de construction en béton pour déterminer leurs aptitudes à servir comme des méthodes étalon d'essai. Ces procédés se différaient entre eux principalement par les conditions de séchage des spécimens, et on les a désignées par RT-50 (73degF et 50 pourcent d'humidité rélative, RT-30 (73degF et 30 pourcent d'humidité rélative), "Modified British" (système anglais modifiè) (122degF et 17 pourcent d'humidité rèlative), et "Rapid" (220-235dgF). A part ces modifications des conditions du séchage, la forme et les dimensions des échantillons ont été variées. L'essai de la retraite au séchage se déroulait en applicant tous les quatre procédés à des briques de sable et gravier, de cendres, de cendres dilatées, de schiste dilaté et d'agrégats de ponce.

Vergleich von vier verschiedenen Methoden zur Bestimmung der Trockenschrumpfung von Beton-Baueinheiten

Vier verschiedene Verfahren zur Bestimmung der Trockenschrumpfung von Beton-Baueinheiten werden verglichen, um ihre Eignung als moegliche Standard-Pruef-methoden festzustellen. Die Pruefungsverfahren unterschieden sich hauptsaechlich durch die Bedingungen, unter denen die Muster getrocknet wurden, und wurden mit den Bezeichungen: RT-50 (73 Grad F. und 50 Prozent relative Luftfeuchtigkeit), RT-30 (73 Grad F. und 30 Prozent relative Leftfeuchtigkeit), Modified British (122 Grad F. und 17 Prozent relative Luftfeuchtigkeit) and Rapid (220-235 Grad F) versehen. Ausser der Aenderung der Trocknungsbedingugen wurden auch die Groesse und die Form der Prueflinge geaendert. Der Trockenschrumpfungs-versuch wurde mit allen vier Verfahren an Sand- und Kieselbloecken, die einer Nachbehandlung im Autoklaven und mit Niederdruckdampf ausgesetzt wurden, an Asche, Streckschlacke, Streck-schiefer und Bimsstein-Zuschlagstoffen ausgefuehrt.

Title No. 58-8

Supporting Structure for Retractable Roof of the Pittsburgh Public Auditorium

By EDWARD COHEN and H. REY HELVENSTON

Paper describes concrete supporting structure, design conditions, construction of the ring girder and podium, foundation, abutments and anchorages, and the concrete requirements. Unique feature of the auditorium is its vast retractable roof which is mounted on a circular reinforced concrete ring girder 34 ft above the arena floor. All concrete has been placed and the Auditorium is scheduled for completion late in September 1961.

■ THE PITTSBURGH PUBLIC AUDITORIUM which is scheduled for completion in September is the key structure in Pittsburgh's Lower Hill Redevelopment (Fig. 1). The construction site as it appeared in November, 1960, is shown in Fig. 2. The construction (see Fig. 3), including mall and parking area, will occupy a 20-acre site.

The auditorium will be constructed to house a wide range of facilities (Fig. 4) including a convention hall, an open air amphitheater, a sports

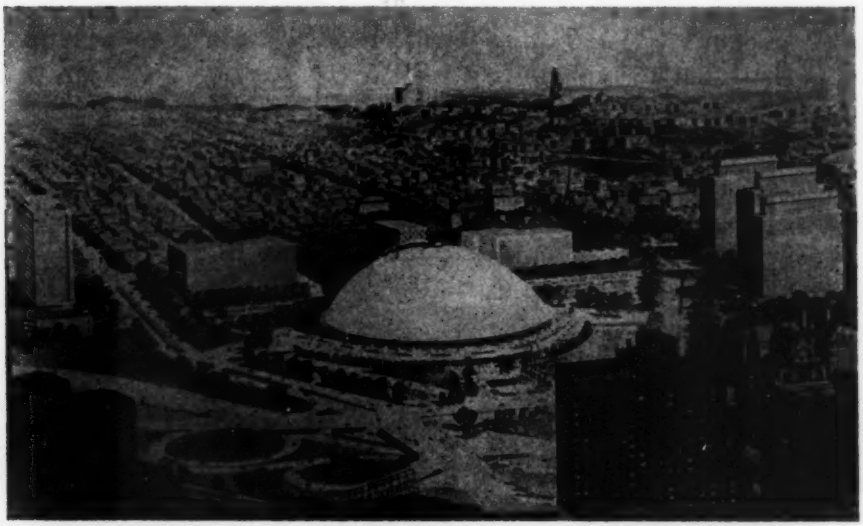


Fig. 1—Architectural rendering of Pittsburgh's Lower Hill Redevelopment

ACI member **Edward Cohen**, an associate with Amman and Whitney, consulting engineers, New York, was project manager of engineering for the retractable roof and supporting structure described in this article. Mr. Cohen joined Amman and Whitney in 1949 where his work has been concerned with the design of special structures and projects. He is the author of numerous papers and a previous recipient of the Wason Medal (1956). He is chairman of ACI Committee 340, Ultimate Strength Design Handbook, and a member of Committees 318, Standard Building Code; 326, Shear and Diagonal Tension; and 328, Limit Design.

H. Roy Helvenston is resident engineer of Public Auditorium Authority of Pittsburgh and Allegheny County, Pittsburgh, Pa. Since 1927 he has had wide experience as field engineer, estimator, construction manager, and coordinator of project scheduling of over 60 projects in different parts of the world. Between 1958-1961 he was in direct charge of the many phases and construction techniques used in Pittsburgh's retractable roof auditorium.

arena, and an exhibit center. Among its outstanding features, the auditorium will provide air conditioning, excellent acoustics, extensive stage accommodations, an ice rink, modern lighting, sound and visual effects, and facilities for both radio and television broadcasts. Embodying an unusually adaptable seating plan, the structure will accommodate from 7500 to 13,600 people depending on the event. Three lots adjacent to the building will provide approximately 1700 parking spaces. Other parking lots in the immediate downtown vicinity will accommodate thousands of additional cars.

The unique feature of the auditorium is its vast retractable roof. The first such dome ever built, the movable roof makes possible a weatherproof auditorium that can be converted to a spectacular open air stadium at the press of a button. One of the largest clear span roof structures in the world, the dome-like roof is nearly circular in plan, with a maximum diameter of 417 ft and a rise (average of all leaves) of 109 ft. The entire roof structure is mounted on a circular reinforced concrete ring girder 34 ft above the arena floor.

The roof is divided radially into eight 45-deg sections, six movable and two stationary. When the roof is retracted, the six movable sections (three on each side) will glide one over the other on top of the two fixed sections to open the huge arena to the skies. It will take 2½ min to move the roof sections from closed to open position.

Each leaf (See Fig. 5) uses seven 30 in. rolled steel beams as ribs

Fig. 2—Construction site in November, 1960

Fig. 3 — Architectural rendering of Pittsburgh Public Auditorium with roof leaves in retracted position

with 10 and 8 in. beam purlins. The outer face of the roof is built up of cellular metal decking covered with rigid insulation, and sheathed with stainless steel. The underside of the framework is covered with a baked enamel zinc coated perforated steel ceiling, enclosing a 2 in. thickness of fiber glass for acoustical absorption.

The six movable leaves differ slightly in radius so that they may be nested. The leaves are supported on spherical bearings at the crown and are mounted on wheels and steel rails around the base. The leaves are designed to roll into the nested position like curved multiple leaf rolling doors. Each of the six movable leaves will weigh about 300 tons.

Four of the six rolling leaves will be mounted on 12 wheels each (five intermediate two-wheel trucks and two single-wheel trucks at the ends). On the other two (upper) rolling leaves one of the end trucks will be of the two-wheel type. All six movable leaves are to be provided with their own motors and brakes. The top leaves, the pair that have the greatest amount of travel, will move at a maximum speed of approximately 4 ft per sec.

The design of the roof and its controls will permit it to be opened or closed against frictional and inertial forces and a wind of about 60 miles per hr.

The dome will have no interior supports. All eight leaves will be supported at the crown by an exterior steel frame that cantilevers from

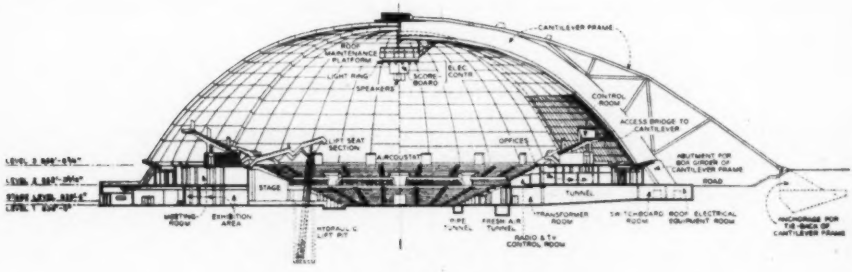


Fig. 4 — Auditorium cross section

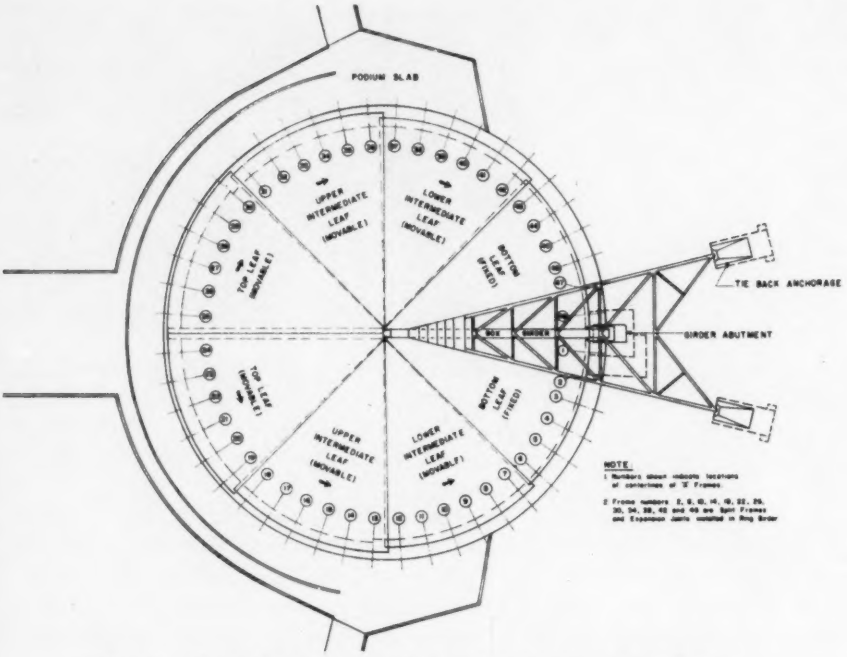


Fig. 5 — Key plan of retractable roof and supporting structure

outside the dome. The exterior steel frame (see Fig. 6) terminates at the top of the dome in a 10 ft cross member. Each end of the cross member is a multiple clevis with vertical pins to which the spherical bearings for four dome sections are connected. The distance between the clevises produces a 10-ft separation between two semi-circular halves. This strip is roofed by 5-ft extensions of the upper leaves on each side.

The cantilever frame which is essentially a curved tripod, is composed of a curved box girder approximately 8 ft wide, 17 ft 6 in. deep, with a system of tieback members extending from the anchorage point near the ground line up to near the top of the box girder. The tieback members and the box girder are interconnected by welded joints to form a triangular space frame which weighs over 1400 tons.

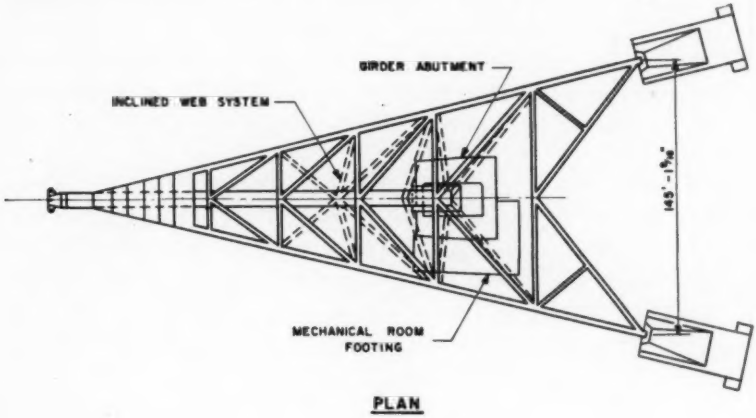
CONCRETE SUPPORTING STRUCTURE

Resting on 48 concrete A-frames (see Fig. 7), the reinforced concrete ring girder (Fig. 8) supports the three sets of rails (Fig. 9) on which ride the six movable leaves partially making up the dome of the arena. In addition, a fourth rail is supported by the ring girder for two temporary rolling scaffolds used during ceiling erection and electrical installation. The ring girder which completely encircles the arena is divided

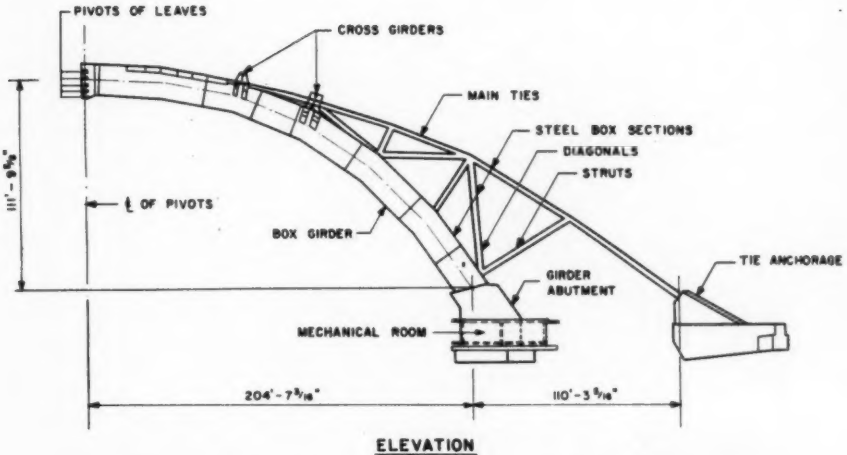
into 108 ft sections of four span rigid frames, the A-frames serving as the vertical legs. One in. expansion joints and double columns are located at the junction between the continuous sections.

The two stationary leaves, which together cover a quarter of the arena area, are also supported by the ring girder. Each rib of the fixed leaves is anchored to the girder with four 2 in. anchor bolts. The girder is about 20 ft in width, except in the quarter supporting only the two outermost (top) movable leaves. Here it is about 16 ft wide and will support a single set of tracks for the top dome leaves.

The upper surface of the ring girder is banked about 13 deg from the horizontal toward the interior of the auditorium.



PLAN



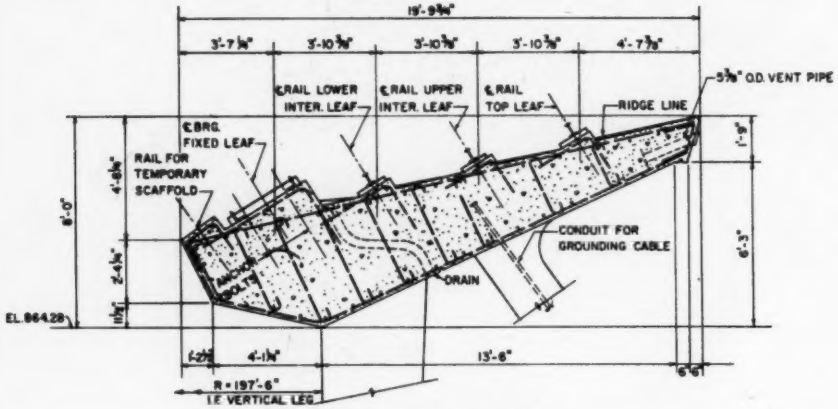
ELEVATION

Fig. 6 — Cantilever frame supporting leaves at their pivot points

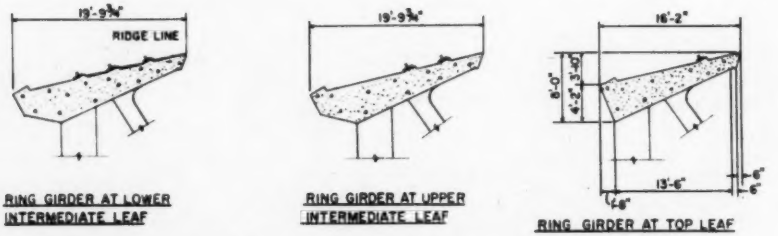
Fig. 7—Reinforced concrete supporting structure under construction in July 1959

The rails, which are parallel to the bottom tangent of the leaf ribs and the wheels, are mounted on continuous haunches superimposed on the ring girder.

The A-frames (Fig. 10 and 11) which function as supports for the ring girder each consist of a vertical and an inclined column. The vertical column has a 2-ft width and a depth varying from 4 ft at the top to



TYPICAL GIRDER CROSS SECTION



RING GIRDER AT LOWER INTERMEDIATE LEAF

RING GIRDER AT UPPER INTERMEDIATE LEAF

RING GIRDER AT TOP LEAF

Fig. 8—Cross sections of reinforced concrete ring girder

Fig. 9 — Ring girder with rails for retractable leaves; forms for next section appear in background

2 ft 10¼ in. at grade level. The 2 x 2 ft inclined column ties back into either a reinforced concrete pier or footing depending on the elevation of the foundation. These piers, which are about 2 ft wide (except at the expansion joints where the total thickness is increased to 2 ft 7 in.) are founded on spread footings on shale. The 2 ft 7 in. dimension is for a double column made up of two 1 ft 3 in. x 2 ft sections plus a 1 in. expansion joint. The lengths of A-frame vertical legs vary from 45 to 14 ft, depending on adjacent construction and the depth to sound bearing. Two-thirds of the piers for the inclined legs also carry the vertical columns that support the center of the podium deck.

The reinforced concrete podium deck consists of a 10-in. concrete slab, about 50 ft wide, that is supported by columns running along the exterior, the center, and the inside perimeter. In some places the exterior edge is supported by a high concrete wall which is circular in plan and battered. The outside diameter of the podium is approximately 500 ft.

The base for the box girder abutment (see Fig. 12) is approximately 42 x 44 ft and has a thickness of 9 ft. The pedestal is 32 x 16 x 34 ft. It is

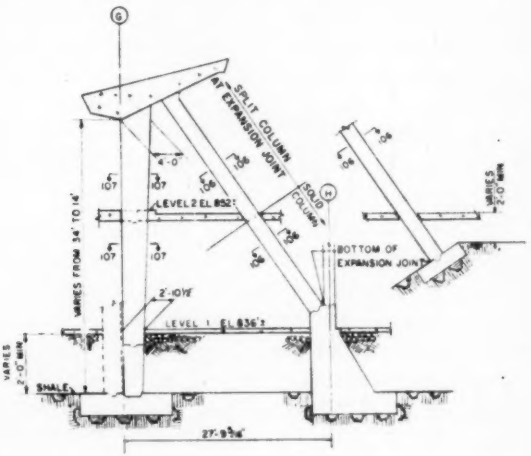


Fig. 10 — Elevation of A-frames supporting reinforced concrete ring girder

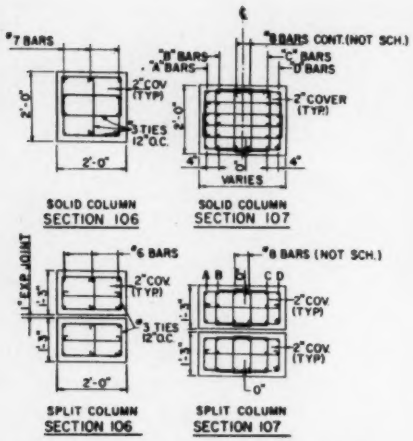


Fig. 11 — Sections through A-frame supporting reinforced concrete ring girder

shaped with a slight bend near the top to accommodate the cantilever girder connection. The box girder is extended into the abutment with shear connectors and interlaced with reinforcement (see Fig. 13) to provide positive anchorage. There are numerous electrical conduits and cells for the wiring which leads through the abutment and passes up the cantilever girder to the pivot.

The base block for each tieback anchorage (Fig. 14) is 56 x 25 ft, and has a maximum height of 19.0 ft. The truncated prism resting on top of the block is about 16 ft high. Twenty-four 2½ in. round prestressed high strength anchor bolts are embedded in the block for the purpose of tying down the tension chords of the space frame of the cantilever girder (see Fig. 15). A shear plate is embedded in the concrete at the junction between the main tie and the anchorage. Guide plates were used for maintaining the reinforcing bar layout and spacing during concrete placement.

DESIGN CONDITIONS

All reinforced concrete of the supporting structure was designed in accordance with the ACI Building Code (318-56) including the Appendix on ultimate strength. Except for hard grade steel in certain A-frame legs, intermediate grade billet steel was used throughout. The specified 28-day cylinder strength, f_c', was 3750 psi and used throughout except for foundations where 3000-psi concrete was specified.

For the ring girder two-thirds of the transverse shear plus all the torsional shear was assumed to be taken by the web reinforcement.

A reaction of 60 tons per wheel parallel to the roof tangent is applied to the ring girder in addition to its own dead weight when the roof is closed and subjected to the design snow load of 30 lb per sq ft on the horizontal projection. The wheel reaction component normal to the rail has a maximum of approximately 4 tons under full load. Under operating conditions, the wheel reaction is reduced to the dead weight component of 35 tons. The traction forces add longitudinal forces of approximately 5 to 10 tons per leaf depending on the particular leaf and

the wind condition. End bumpers were designed to carry a force of 20 tons per leaf under malfunctioning of the control and braking system.

Temperature changes of ± 20 F and shrinkage equivalent to a temperature drop of 30 F were included in the analysis.

For the roof closed and the roof open positions the wheel loads are in a known fixed position. However, during roof operation the relative positions of the wheels on the overlapping leaves vary. Because the rails are spaced across the 20-ft width of the girder and the various leaves can move independently of each other the resultant of the wheel loads does not pass through the shear center of the girder under many possible wheel distributions and torsion results. To facilitate the analysis, influence coefficients for moment, thrust, shear, and torsion were computed and used to evaluate the maximum forces caused by the moving loads at all sections.

The A-frames supporting the ring girder are subject to bending in their own planes due to eccentricity of horizontal and vertical components of the girder reactions with the centroids of the legs and due to the change in length of the legs due to the axial forces. Also, they are subject to simultaneous moments normal to their planes as a result of traction, temperature, shrinkage, and the vertical loads acting on the ring girder.

The legs of the split buttresses at the expansion joints are only 1 ft 3 in. wide. Each leg was joined into a single solid 2 ft 7 in. leg below the

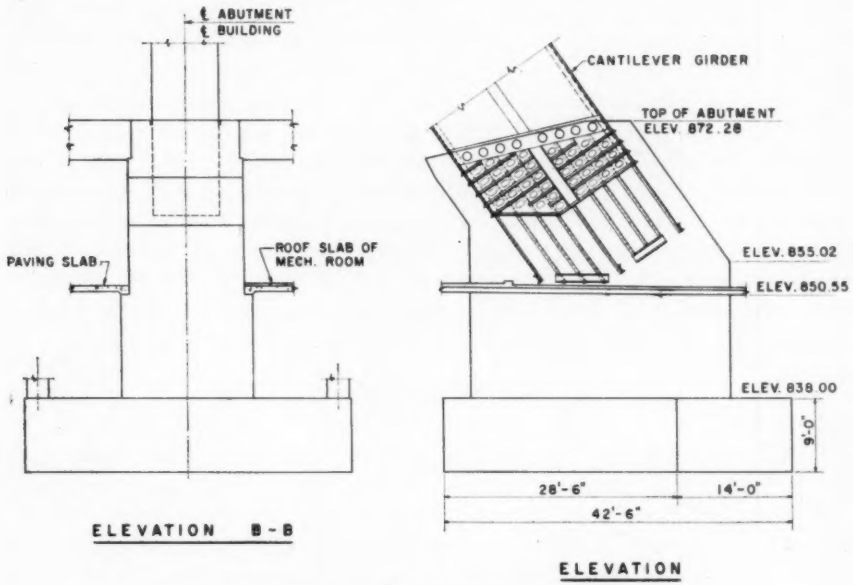


Fig. 12 — Elevations of cantilever girder abutment

Fig. 13 — Shear connectors and reinforcement at girder abutment

podium level. This provides adequate flexibility to minimize temperature and shrinkage effects without reaching excessive L/D ratios.

When the roof is open each tieback anchorage is subject to a maximum tension in the direction of the structure steel of about 2000 tons. When the roof is closed with a snow load of 30 lb per sq ft on one half the roof a tension of 1200 tons and a transverse shear of 270 tons act simultaneously. A factor of safety of 1.25 was maintained against overturning neglecting the effect of the earth backfill. Considering the backfill the factor of safety is greater than 2.0. The bearing surface was placed against hard shale with an allowable bearing capacity of 8 tons per sq ft. The depth of bearing was determined by the slope of the strata and the roadway and trench excavation between the anchorages and the main abutment. The tension from the cantilever frame is transferred by twenty-four 2½ in. diameter high strength bolts at each anchor. The 21-ft bolts are coated with mastic and anchored in a structural steel frame. They were prestressed to a total stress of 125 tons each prior to decentering the frame. The tension in the bolts was checked by the change of length of the bolts as well as by strain measurements above the concrete.

The buttress for the box girder is designed for similar factors of safety as described above for the anchorage. In addition to its own dead weight it is subject to a thrust of 3400 tons combined with a moment of 10,000 ft-tons and a shear of 750 tons in the plane of the girder when the roof is open. When the roof is closed there are no forces due to roof dead load. However, when a snow load on half the roof is considered, the thrust becomes 2000 tons; the moment becomes 1000 ft-tons; the shear becomes 300 tons; and a torsion of 750 ft-tons is introduced.

CONSTRUCTION

Ring-girder

Prefabricated timber forms supported on steel scaffolding were used for ring girder construction. Two complete sets of forms were built, intended for seven uses. Since the 20 ft wide banked girder has an irregular cross section, with a maximum depth of about 5 ft, the plywood forms consisted of several conical surfaces; inner and outer fascia; and two bottom slopes. They were supported at 16 in. intervals on radial

wood joists. Wood trusses transmitted the load from these joists to 2 x 8 in. laminated wood beams along the curve. Shores were tubular steel scaffolding.

The forms had to be set accurately to the required elevation and alignment. Where the shores could not be supported on the podium deck, granulated slag was spread on the ground, which was roughly graded, to level up under the shore sills. These shores were about 36 ft high.

The slope of the top surface of the girder was flat enough to be shaped without forms. However, the four banked annular haunches which carry the rails required forms.

To insure uniform setting of the 5000 sloping bolts for the rails and the haunches, the steel contractor furnished eight template frames. The eight frames bolted side by side made a four bay casting section. They were supported on the radial joists with unequal-length angle posts, to obtain the desired slope. Curved wood forms for the haunches and templates for the bolts were attached to the bottom of steel channels extending below the frames. The bolt templates consisted of waxed paper sleeves set inside pipe spacers welded to steel plates, which in turn, were welded to the channels.

Ring girder forms supported a loading of approximately 7 tons per lineal ft of ring girder. It was necessary to exercise considerable care in placing the sills supporting the scaffolding and in frequently jacking the scaffolding frame. During the placing of all structural concrete supported by metal scaffolding, a crew of carpenters was continually engaged in checking the loads transmitted to the scaffolding and in tightening scaffold jacks. Also they constantly checked all wood and timber bearings in the forms, rectifying minor settlements as they were discovered. Two men were engaged in continuous readings throughout the average 6 hr of each ring girder placement of approximately 250 cu yd. The purpose of these readings was to certify that no excessive settlement occurred.

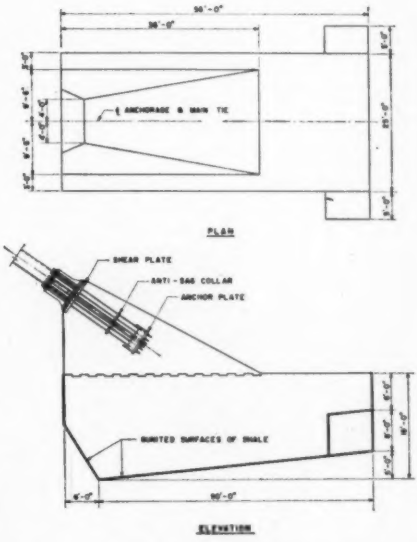


Fig. 14 — Plan and sectional views of tieback anchorage

The girder was concreted in four-bay lengths with one crane and bucket at each end of the placement (Fig. 16). The concrete was placed first at the ends of the sec-

Fig. 15 — Tension chords of cantilever girder space frame connected to tie-back anchorages

tion, then worked toward the center. A pattern of settlement readings was produced, indicating vertical displacement at each end with a "hump" in the center of the forms before placing concrete there. To avoid occurrence of these displacements in the forms for subsequent placements, the concrete was first placed in the center of each "four bay grouping" and then extended toward the ends. This procedure eliminated the problem of settlement, which was felt to be attributable to the continuity of the form system. A plan indicating the concrete placing sequence is shown in Fig. 17.

The first element of the form to be stripped was the inside fascia. It was moved separately and replaced on the interior of the reset soffit sections after these had been properly positioned in their new locations. The exterior fascia was permanently attached to the soffit form. The forms were either slid or rolled into and out of their positions from the outside of the ring girder location to avoid wedging and to receive the support of the podium slab on grade. Cranes moved the forms, while trucks handled the steel scaffolding.

Podium

The horizontal 10 in. podium deck was formed with ⅝ in. thick 4 x 8 ft plywood panels. They were supported on 2 x 4's spanning between adjustable-length steel beams. The relatively large span of these beams reduced appreciably the number of rows of 4 x 4-in. shores needed. Not only was this arrangement advantageous in eliminating a forest of lumber, but it avoided a great deal of tedious work in careful installation of shoring sills to prevent washouts in heavy rain.

Fig. 16 — Ring girder concreted with one crane and one bucket

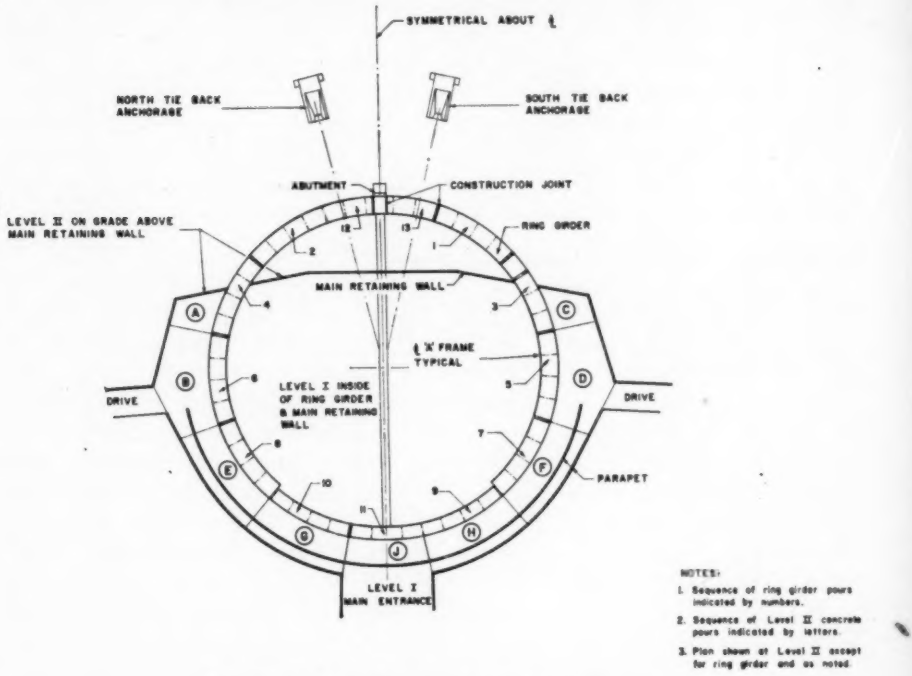

SYMMETRICAL ABOUT ℄
NORTH TIE BACK ANCHORAGE
SOUTH TIE BACK ANCHORAGE
ABUTMENT
CONSTRUCTION JOINT
LEVEL II ON GRADE ABOVE MAIN RETAINING WALL
RING GIRDER
MAIN RETAINING WALL
℄ 'X' FRAME TYPICAL
LEVEL I INSIDE OF RING GIRDER & MAIN RETAINING WALL
DRIVE
DRIVE
PARAPET
LEVEL I MAIN ENTRANCE
NOTES:
1. Sequence of ring girder pours indicated by numbers.
2. Sequence of Level II concrete pours indicated by letters.
3. Plan shown at Level II except for ring girder and as noted.

Fig. 17 — Plan of concrete placing sequence

The plywood sections were covered with plastic paint after each use so that the 4 x 8-ft sheets could be re-used. Concrete placement for all vertical walls was done in steel-backed plywood panels.

It was necessary to provide sufficient support for the podium deck to resist the load placed above it during the construction of the ring girder. Additional steel scaffolding rested on the podium deck, supporting the forms for the higher lift, as well as the dead weight of the concrete being placed. Care was exercised during the forming and placing of the podium deck to allow for passage of the inclined ring-girder column which passes through an opening in the podium deck.

The ready-mixed concrete for the podium deck and other sections of the structure was placed by one of the cranes. After the concrete had been placed for the podium deck, work was ready to start on the ring girder just above. After podium deck forms and shoring were stripped by the contractor, a tubular steel scaffolding was installed as reshoring.

Concreting had to be carried out regardless of cold weather, when many cycles of freezing and thawing could be expected. To preheat the reinforcing bars and forms and to keep the fresh concrete warm, oil

burning salamanders were inserted under the formwork inside an enclosure formed by hanging tarpaulins around the outside. Insulation blankets covered with tarpaulins protected the top surface of the concrete. The natural movement of heated air was always permitted. Temperatures were carefully observed with special arrangements to assure temperature examinations of the concrete and test cylinders over weekends. High-early-strength cement was used in all ring girder and podium slab placements to release forms quickly for re-use.

Foundation, abutments, and anchorages

Foundations for the easterly A-frames above the main wall were in place by the first part of August, 1958. The remainder were cast concurrently with the podium slab and ring girder, being completed in May 1959.

The excavation for the abutment of the box girder of the space frame supporting the roof leaves encountered soft brown shale which gave the appearance of a stratified blue rock at first glance. However, it was noticed that the surface of the blue shale softened slightly after some time when exposed to moisture. To prevent this undesirable plasticizing reaction, the bearing surfaces of the shale were cleaned, and a ½ in. layer of shotcrete was promptly applied.

Simultaneously, excavations for the tieback anchorages (Fig. 18) were being made and prepared to receive the concrete. Each anchorage was designed to resist a pull of 2100 tons (Fig. 19). The surfaces required to resist maximum bearing, and also the shear keys, were given the same shotcrete treatment applied to the abutment foundation.

Vertical panels extending up to the brown shale were used in the casting of the abutment. There was no necessity to bridge over the excavation to construct forms or to cast concrete. However, in the case of the tiebacks, the major portion of the formwork for the bottom and top placements was sloped. In addition, there were numerous long large diameter reinforcing bars bent to conform to the sloped outlines of the foundations. These had to be embedded within the bottom and top placements. The excavations were bridged with old telephone poles removed earlier from the site. These poles served as support for elephant trunk hoppers used in casting the concrete, for walkways, and for the posi-

Fig. 18 — Excavation for tieback anchorage with spacer plates in place

tioning of the forms, steel frames, and reinforcing bars.

Fig. 19 — Tieback anchorage in place

The second and final placement of each of the tieback anchorages required "backforming" the face which provided easiest access for casting due to the existence of the shale banks close to the other sides. Also adding to the complexity was the necessity of maintaining the alignment and location of the fabricated frames which gave support to reinforcing bars, heavy shear plates, and the twenty-four 2½ in. diameter bolts which were encased to anchor the space frame supporting the roof. The final position of the top plate with its sloping and exposed upper face had to be correctly positioned relative to each direction and slope. Helping to accomplish this was the installation of a cable with turnbuckles and twisted No. 9 wire, extending in several directions and anchored to dead men.

In the final abutment placements, the upper end of the two cells, enclosed by the sloped steel box girder which was to be embedded, represented the only space inside the forms which would not be concreted. The reinforced concrete walls which surrounded the embedded girder contained several layers of large reinforcing bars (plus transverse reinforcing through anchor holes in the lower portion of the encased girder). Therefore, the vibration was carried out by lowering vibrator hoses through the spaces between the bars. Eight inspectors were assigned to guard against the formation of concrete voids.

As the final grade of the abutment and the surface of the ring girder beyond the embedment were the same, special care was required in finishing the concrete after the last placement. In addition to the slopes of the top of the abutment and of the side recesses for ring-girder bearing, there was the requirement of matching the curved tops of the ring-girder sections. Running parallel to and lying within the embedded double box girder were twenty-four 5 in. diameter conduits for the electrical cables of the roof. They were spaced on 2-ft centers in one direction and 3-ft centers in another direction.

The sequence of abutment placements consisted of (1) 665 cu yd, (2) 244 cu yd, and (3) 309 cu yd. The two placements of each of the tieback anchorages amounted to (1) 863 cu yd, and (2) 196 cu yd. The use of larger sized aggregate for the abutment concrete was governed by the desire to attain a maximum density and minimize shrinkage.

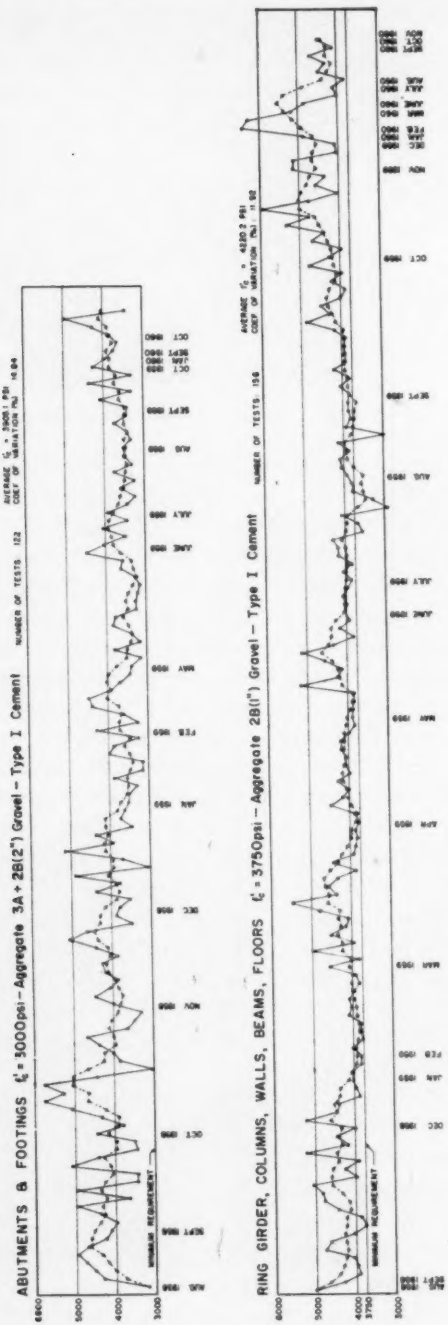

ABUTMENTS & FOOTINGS f'c = 3000psi – Aggregate 3A + 2B(2") Gravel – Type I Cement
RING GIRDER, COLUMNS, WALLS, BEAMS, FLOORS f'c = 3750psi – Aggregate 2B(1") Gravel – Type I Cement
RING GIRDER f'c = 3750psi – Aggregate 2B(1") Gravel – Type III Cement
CONCRETE MIXES
f'c = 3000PSI
f'c = 3750PSI

Fig. 20 — Concrete cylinder strength test data

This was accomplished despite the difficulty in placing the concrete in the heavily reinforced forms.

CONCRETE STRENGTH AND MIX REQUIREMENTS

In establishing the cement content and aggregate quantities preliminary mixes were made in the laboratory with each size of aggregate to develop a strength curve. From the plants previous experiences it was believed that the coefficient of variation for the 3000-psi concrete might be expected to be approximately 12 percent and for the 3750-psi mix about 10 percent. To achieve 99 chances in 100 of the tests meeting specifications it was necessary, therefore, to provide concrete of 140 percent of the required strength in the 3000- or 4200-psi concrete, and 130 percent of the required 3750- or 4875-psi concrete. From the strength curves it was established that for 3000-psi concrete using 2B and 3A gravel a 5-sack mix could be expected to provide 4660 psi. For 3000 psi requirements with 2B aggregate a 5-sack mix would provide 4730 psi; for 3000-psi concrete with No. 2 aggregate 5.00 sacks would provide 4380 psi. The 3750-psi concrete using 5.40 sacks could be expected to provide 4830 psi or 128.8 percent of the required strength.

The city of Pittsburgh building code requires a minimum of 5.5 sacks of cement per cu yd for concrete exposed to weather or ground and a minimum of 5.0 sacks for concrete protected from weather and ground. The work on mix proportioning indicated that, under adequate supervision and control, the required strengths could be obtained with a lower cement factor than the 5.5 required by the code. Permission was obtained from the Bureau of Building Inspection to use the cement factors indicated on the design curves, but not less than 5.0 sacks provided that adequate suitable controls would be set up and maintained.

During the progress of the project all test results were plotted and a running average of the five previous tests also plotted (Fig. 20). After 13 months the cement contents were increased slightly to provide additional protection against some of the low strengths obtained.

The aggregate sizes used were: a combination of 3A and 2B gravel to provide a nominal 2-in. aggregate; 2B gravel to provide a nominal 1-in. aggregate; and No. 2 gravel to provide a nominal ¾-in. aggregate.

The coefficients of variation and average strengths are shown in Fig. 20. Shown also are the mix quantities as originally established and as revised.

ORGANIZATION

The owner of the Pittsburgh Auditorium is the Public Auditorium Authority of Pittsburgh and Allegheny County. Edward R. Fraher is executive secretary and H. Rey Helvenston is the resident engineer and general superintendent of construction for the nine prime contracts on the auditorium.

The architects of the project are Mitchell and Ritchey of Pittsburgh.

The engineering design of the Pittsburgh Public Auditorium retractable roof and supporting structure, including structural, mechanical, and electrical features, was done by Amman and Whitney, consulting engineers, New York. Boyd G. Anderson was the partner in charge of design. Werner Amman was the partner in charge during the construction phase. Edward Cohen was project manager throughout the project. Alan Anderson was project engineer for the supporting structure with Michael Fiegen, Nan Sih, and other members of the staff participating in the final design of the reinforced concrete supporting structure.

R. A. Zern is the consulting structural engineer on the interior concrete construction other than the roof supporting structure and the podium.

Dick Corp. is the prime contractor responsible for all the concrete construction. Joseph Alcorn is the general superintendent for this contractor.

ACKNOWLEDGMENTS

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American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

Discussion of this paper should reach ACI headquarters in triplicate by Nov. 1, 1961, for publication in Part 2, March 1962 JOURNAL

Estructura de soporte para el techo retráctil del Auditorio público de Pittsburgh

El documento describe la estructura de soporte de concreto, las condiciones de diseño, la construcción de la viga circular y del podio, la fundación, los estribos y los anclajes, y los requisitos del concreto. La característica distintiva del auditorio es su techo retráctil enorme que está montado en una viga circular reforzada de concreto a 34 pies por encima del suelo. Todo el concreto se ha colocado y el auditorio ha de terminarse a fines en septiembre de 1961.

Structure de soutènement pour le toit escamotable de la Salle Publique de Pittsburgh

Ce rapport décrit la structure de soutènement, les conditions de la conception, la structure de la poutre annulaire et la plate-forme circulaire en soubassement, la substruction, les butées et leurs ancrages, et les nécessitiés de béton. Le trait saillant de la salle est son vaste toit rectangulaire escamotable monté sur une poutre annulaire en béton armé, 34 pieds au-dessus du plancher de l'arène. Tout le béton est coulé et la Salle sera prête vors la fin en septembre de 1961.

Traggestell fuer einziehbares Dach fuer das oeffentliche Pittsburger Auditorium

Die Arbeit beschreibt das Beton-Traggestell, die Konstruktionsbedigungen, die Ausfuehrung des Ringtraegers und des Podiums, des Fundaments, der Widerlager und Verankerungen, und die Anforderungen an den Beton. Das hervorstechende Merkmal des Auditoriums ist das ungeheure einziehbare Dach, das auf einem kreisformigen Eisenbeton-Ringtraeger 34 Fuss ueber dem Boden montiert ist. Der Beton ist bereits gegossen, und die Fertigstellung wird im September 1961 erwartet.

Title No. 58-9

Characteristics of Sorption and Expansion Isotherms of Reactive Limestone Aggregate

By R. F. FELDMAN and P. J. SEREDA

Characteristic differences have been detected in the sorption and expansion isotherms of alkali-treated and untreated reactive limestone aggregate; these results are compared with those obtained from Vycor glass under similar conditions. It is concluded that the evidence establishes the presence, within the pores of the aggregate, of trace amounts of a material that causes expansion when water is made available to it and that the mechanism of expansion is similar to the alkali-silica complex formed in the pores of Vycor glass although the composition of the materials in the two cases may be different.

■ EXPANSION OF CONCRETE AGGREGATE in the presence of a high alkali cement has been studied extensively and various hypotheses have been proposed to explain the mechanism.[1,2] The classical alkali-silica reaction has been characterized by the presence of the "alkali-silica complex" gel and Powers and Steinour have proposed an hypothesis for this system.[1]

Recently it has been found that dolomitic limestone from Kingston, Ont., Canada, when used as coarse aggregate in high alkali cement, causes expansion and cracking of concrete.[3,4] To date it has not been possible to account for the expansion either by gel formation since the presence of gel has not been proved, or by the mineralogical changes such as the dedolomitization which occur during alkali treatment. Dedolomitization is the process by which the dolomite is replaced by calcite and brucite.

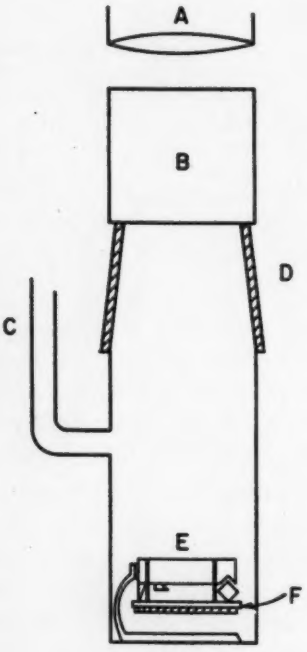
It has now been found in this laboratory that these limestones exhibit a significant change in their sorption and expansion isotherms due to treatment with alkali and it is proposed that these changes are characteristic of such systems and may shed some light on the mechanism of the reaction. Results of sorption and expansion isotherms for a number of horizons of the Kingston limestone before and after alkali treatment are here discussed and compared with corresponding results obtained for Vycor glass.

EXPERIMENTAL

Treatment of materials

Limestone—Rectangular wafers, 3 x 1 cm, and 1 to 2 mm thick, were cut out with a diamond saw from alkali-treated and untreated prisms previously used by Swenson and Gillott.[4] These prisms had been selected from beds of one of the operating quarries at Kingston and occur at depths of 6-7, 10½-12, 20-21, and 24-30 ft from the surface. The reactivity of these specimens is described in detail in Reference 4. Samples of nonreactive Ottawa Valley limestone, obtained from the same workers, were included in this study.

Vycor glass—Porous Vycor glass, with composition 96 percent SiO_2, 3 percent B_2O_3, and in similar dimensions as the limestone samples was treated for 25 min in 3N NaOH solution. This sample was washed in water but not leached for any length of time until after sorption measurements were made.



A COLLIMATOR
B EVACUATED SPACE BETWEEN TWO OPTICAL PLATES
C TUBE LEADING TO APPARATUS
D GROUND-GLASS JOINT
E EXTENSOMETER
F SAMPLE

Fig. 1—Cell used for expansion isotherm measurements

Sorption apparatus

A high-vacuum apparatus, in which the samples were exposed to water vapor only, was used in this work. Six samples can be kept under test simultaneously. These were usually in the form of thin wafers, and were suspended by platinum hooks from quartz spirals of the McBain-Bakr type. A sensitivity of 2.5×10^{-5} g using a 1-g sample, was attained with a cathetometer reading to 0.001 cm.

Water vapor was introduced to this system from a bulb which was situated in a bath that could be controlled at any desired temperature between 18 and 70 F within 0.1 F. The samples were maintained at 70 F by immersing the lower ends of the tubes containing the spirals to a depth of 12 in. in a controlled bath. Room temperature was controlled at 73 ± 1 F.

On this apparatus was a cell, containing a commercially available extensometer mounted on a sample, which determined the expansion isotherm. Details of the cell and extensometer are shown in Fig. 1.

The sample in the cell was exposed to the same conditions as the samples in the sorption tubes. Readings were taken by optical means to 2×10^{-4} in. per in.

A three-stage oil diffusion pump backed by a rotary vacuum pump was employed to obtain pressures less than 10^{-4} mm Hg.

R. F. Feldman has been engaged in research, since 1959, with the Building Materials Section, Division of Building Research, National Research Council, Ottawa. For the past 2 years he has been studying the sorption properties of reactive limestones and the related dimensional changes.

P. J. Sereda has been engaged in research, since 1948, with the National Research Council, Ottawa. Prior to joining the Research Council he was employed by the Atomic Energy of Canada, Ltd., Chalk River, Ont., where he worked on the treatment of water for the cooling of an atomic pile. Recently he has been organizing work in connection with the sorption of water in porous materials.

Procedure

The procedure adopted varied from sample to sample only in the time allowed for equilibrium to be attained.

Before a sorption or expansion isotherm was started the sample was outgassed at 140 to 150 C at a vacuum of less than 10^{-6} mm Hg until negligible weight loss was recorded during 24 hr. From 3 to 5 hr was found adequate for all untreated samples. For treated samples 24 hr was required.

For equilibrium to be attained between points along the isotherm, 1 to 3 days was generally allowed; 15 hr of negligible weight change always elapsed before the system was considered to be in equilibrium.

In the dimensional change work, outgassing with heating took place in a vacuum system before the sample was mounted on the extensometer. Mounting was performed in a low humidity room while the sample was still warm. This system was then transferred to the cell and re-evacuated. It is considered that this procedure caused only small errors in the expansion measured. Modifications have now been made so that outgassing is carried out on the mounted sample.

The Vycor glass sample with the extensometer mounted on it was immersed in water for several hours, by which time all expansion had ceased. The sample was removed and rapidly immersed in a 3N alkali solution for 25 min, expansion measurements being taken during this period.

RESULTS

Sorption isotherms

Untreated limestone samples—As shown in Fig. 2 to 6, all samples of untreated limestone gave Type II or Type IV isotherms according to the classification of Brunauer.[5] These curves show that the porosities in terms of percent by weight of water held in the sample at conditions approaching saturation, are small. As shown in Table 1, the porosities were all less than 0.5 percent with the exception of the 24- to 30-ft (green) horizon, which had a porosity of 3.37 percent.

Surface areas were calculated by the BET* method assuming that one H_2O molecule occupies an area of 10.8 square Angstroms. As shown in Table 1 they are all less than 8 sq m per g, which is relatively small for porous materials. On the desorption path, a primary hysteresis loop is evident which is generally accounted for by theories based on

*The so-called BET method was developed by Brunauer, Emmett, and Teller. See Reference 5 for complete details.

capillary condensation. The adsorption path is rejoined for these samples at a P/P_0 value of about 0.5.

Treated limestone samples (leached and unleached)—A comparison of the isotherms before and after treatment, as presented in Fig. 2 to 6, show the following characteristic differences.

1. An increase in sorbed water along the isotherm

   Whether this is due to the formation of a compound during the treatment with alkali, resulting in an increase in surface area or due to the formation of some form of hydrate, will be discussed later.

2. An increase in porosity

3. Distortion of the normal hysteresis loop observed in untreated samples

4. Secondary hysteresis

   In this type of hysteresis, discussed by McDermot and Arnell,[6] the desorption part of the loop never rejoins the main curve. This has been observed for materials having large swelling properties such as materials with the characteristics of a gel. Work on hardened portland cement paste by Jesser, Berchem, and Giertz-Hedstroem and more recently by Powers and Brownyard[7] has shown this type of hysteresis in its isotherm. This has also been observed for cellulose and wood products by Stamm,[8] Spalt,[9] and others.

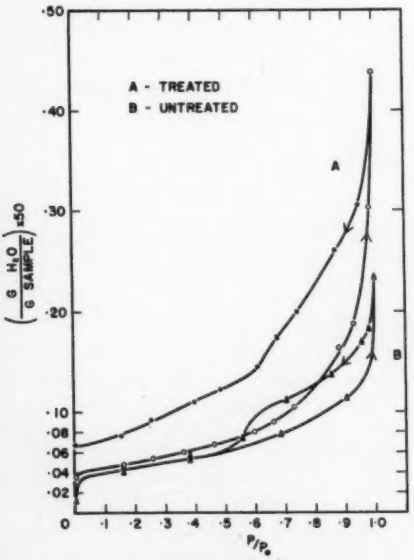


Fig. 2—Sorption on Kingston limestone (6-7-ft bed)

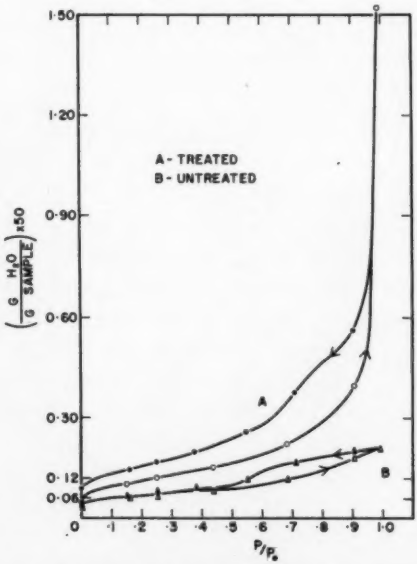


Fig. 3—Sorption on Kingston limestone (10½-12-ft bed)

TABLE I—POROSITY AND RESIDUAL SORPTION RESULTS

Horizon	Porosity, $\frac{\text{g water}}{\text{g sample}} \times 100$		Area, sq m per g		Residual sorption, $\frac{\text{g water}}{\text{g sample}} \times 100$
	Treated	Untreated	Treated	Untreated	
6-7 ft	0.88	0.47	2.75	2.65	0.064
10½-12 ft	3.04	0.42	7.06	4.02	0.090
20-21 ft	1.16	0.36	4.58	4.44	0.040
24-30 ft (green)	5.13	3.37	20.40	8.60	0.234
24-30 ft (grey)	3.17	0.47	8.72	4.31	0.084
Ottawa Valley	0.61	0.42	—	—	0.016
Vycor glass	22.28	20.00	243.00	144.70	2.300
Vycor glass (leached)	6.20		55.50		—

5. Residual sorption

After the sample had been evacuated for a period when negligible weight loss was recorded over 24 hr, the samples were outgassed as described. The weight loss of the sample when it had returned to room temperature was recorded as the residual sorption. In Table 1, residual sorption is given in terms of percent by weight of water held in the sample.

Results for the Ottawa Valley nonreactive limestone showed a small residual sorption and secondary hysteresis, though results in terms of surface area and porosity were not reproducible from sample to sample.

Treated samples of 24-30 ft (green) and 10½-12 ft horizons were leached in water for 5 and 19 days, respectively. These samples yielded results similar to those of the unleached sample both qualitatively and quantitatively. Only the upper portion of the isotherm was affected, producing a slightly lower value for the porosity.

This result is probably because values obtained at conditions approaching saturation are difficult to reproduce because of the critical nature of this condition. As seen later from results on the leaching of treated Vycor glass, the leaching of a readily dispersible material from the pores of a substance takes place quite rapidly. It is thus concluded that free sodium hydroxide is not present in the pores of the limestone in a significant quantity to affect the results of the isotherm either qualitatively or quantitatively.

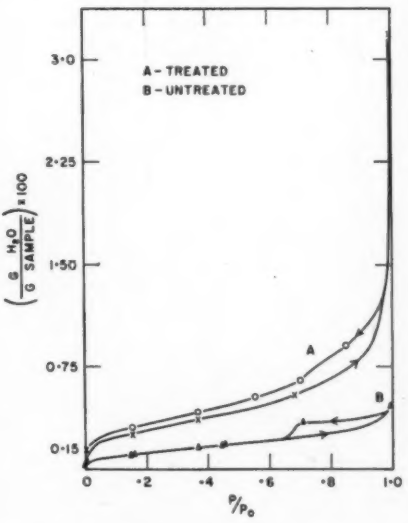


Fig. 4—Sorption on Kingston limestone (24-30-ft bed) grey

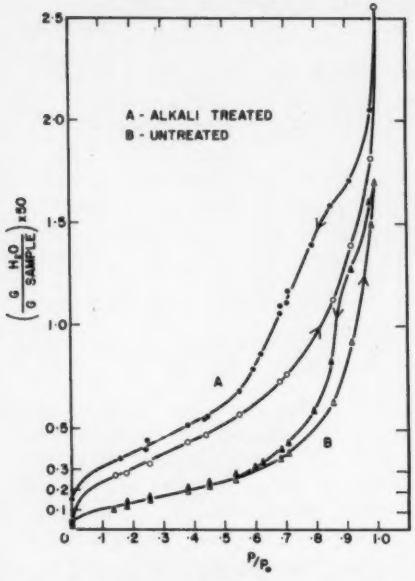


Fig. 5—Sorption on Kingston limestone (24-30-ft bed) green

Calculations of the surface areas of the treated samples were made by the BET method as described. It is stressed, however, that these values are only valid if the sorption occurring is physical and not a form of chemi-sorption or hydration. Table 1 gives a summary of all the information obtained from this portion of the work.

Vycor glass—During alkali treatment of Vycor glass, which was performed after the sample had been saturated with water, expansion occurred at a rapid but decreasing rate as shown on Fig. 7. The isotherm for the treated Vycor (Fig. 8) exhibited similar characteristics as the treated limestone. The sample of treated Vycor glass, which was leached for 24 hr, yield-

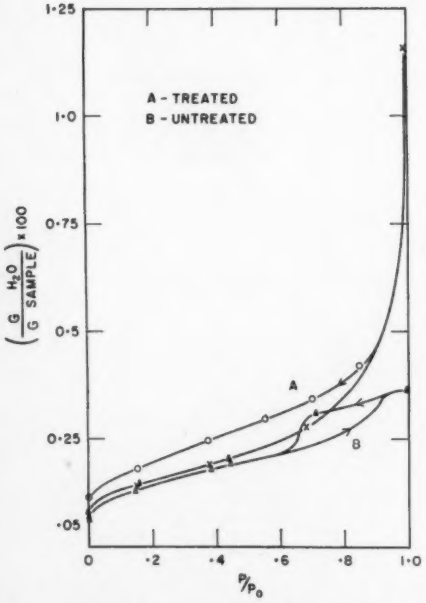


Fig. 6—Sorption on Kingston limestone (20-21-ft bed)

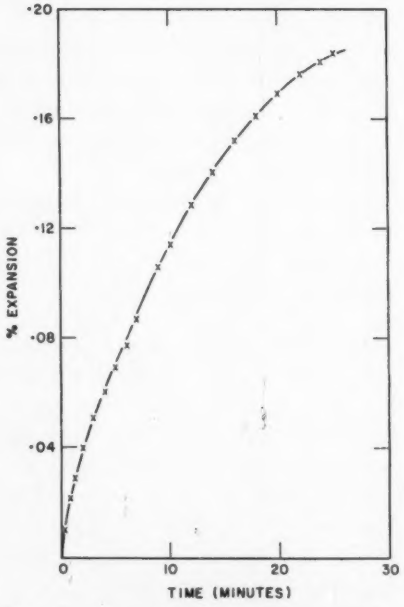


Fig. 7—Expansion of Vycor glass during treatment in alkali

ed an isotherm similar to the untreated sample, but with a greatly reduced surface area. The total water adsorbed at saturation conditions also shows a reduction over that of the unleached sample. It is considered that large pores are formed by the corroding action of the alkali and indeed large cracks can be observed throughout the sample; at conditions approximating saturation these cracks and pores were not filled. In reality, the pore volume of the leached sample should be considerably larger than that of the unleached sample. A summary of these results is also presented in Table 1.

Expansion isotherms

Expansion isotherms were measured for the treated and untreated samples of the 10½- to 12-ft and 24- to 30-ft (green) horizons. The untreated samples of both show low expansion for the entire range of humidity (just less than 0.04 percent as shown in Fig. 9 and 10). Both curves show slight primary hysteresis, but no evidence of secondary hysteresis or residual expansion.

For the treated samples, the curves in Fig. 9 and 10 show characteristics similar to the sorption isotherm curves. A large residual expansion and secondary hysteresis and a more rapid expansion with increasing humidity than with the untreated sample, is evident.

If the expansion of these limestones can be explained by a hypothesis which attributes the expansion to the formation of a material which imbibes water, either a gel or a hydrate, a degree of reversibility should be expected in this expansion when the sample is dehydrated. This seems to be the case with the limestone from the 24- to 30-ft horizon (green) where the total expansion for P/P_0 0-1, is 0.04 percent for the untreated as compared with the 0.12 percent for the treated sample. Assuming that the limestone is in a saturated state during treatment, then the difference 0.08 percent may be considered as the reversible portion of the expansion that occurred during treatment. The treated limestone of the 10½- to 12-ft horizon does not expand more than the untreated, but this may be accounted for by the fact that there was

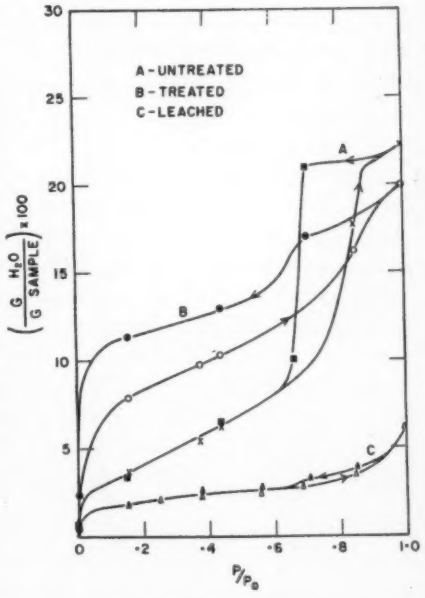


Fig. 8—Sorption on Vycor glass

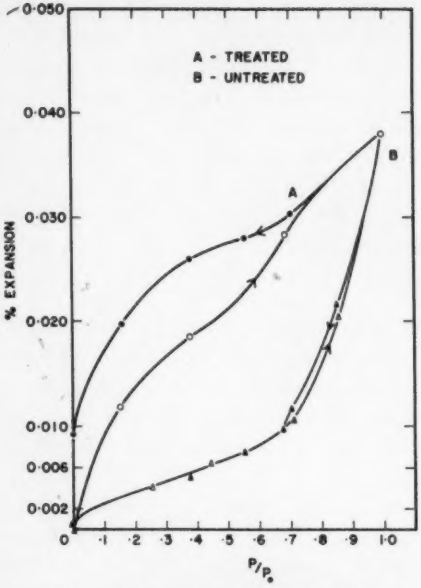


Fig. 9 — Dimensional change during sorption on Kingston limestone (10½-12-ft bed)

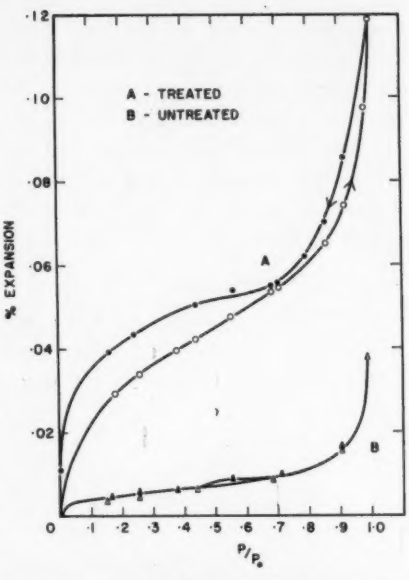


Fig. 10 — Dimensional change during sorption on Kingston limestone (24-30-ft bed) green

a large pore volume increase and a large expansion on alkali treatment.[4] This large pore volume increase may be attributed partially to microcracking throughout the pores thus making the expansion irreversible.

DISCUSSION

Despite the smoothness of the sorption isotherms of the alkali-treated limestones, it cannot be concluded that the increase of sorbed water over that of the untreated samples is due to the formation of a compound which produces an increase in surface area.

It is presumed from previous work on isobars[10] that a mixture containing crystals of various sizes extending into the range of colloidal dimensions would produce a smooth isotherm. Applying this to the case of the alkali-treated limestone, these small crystals would form in a range of sizes within the limited space of the pores of the limestone.

The formation of a compound possessing several hydrates may account for the entire sorption path. It is more difficult, however, to explain the secondary hysteresis and residual sorption by this approach. These characteristics are typical of a sorption system which swells[6] and are explained as follows: when vapor is sorbed by the system, the crystallites swell because of intercrystalline penetration. Once this penetration has occurred, some molecules of vapor are trapped and can only be removed

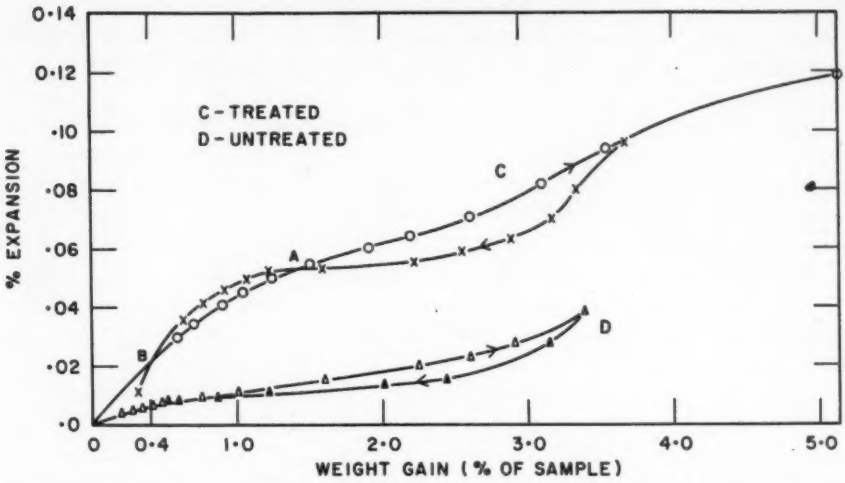


Fig. 11—Expansion versus weight gain for Kingston limestone (24-30-ft green)

by warming the sample and desorbing the vapor. With some gel systems this swelling can occur gradually and continuously over the whole course of the sorption loop. The expansion isotherm of the alkali-treated limestone has shown a residual swelling and secondary hysteresis.

An explanation for the secondary hysteresis based on the formation of a hydrate can be attempted by supposing that some hydrate became trapped within the material during desorption and dehydration; the fact that dedolomitization occurs in alkali-treated expanded rock may well have some bearing on the trapping of hydrate within the rock. Results of a somewhat similar nature have been obtained from the hydration-dehydration of $CaSO_4$.[11]

The hypothesis of Powers and Steinour[1] which explains the expansion of concrete caused by aggregates of high silica content by a swelling gel formation, cannot be disregarded at this stage. This postulation includes the formation of the alkali-silica complex, necessarily of a high surface area, which on adsorbing water creates swelling pressures and thus causes expansion.

It has been shown from expansion results on Vycor glass during alkali treatment (Fig. 7), that pressure within the pores of the Vycor glass build up rapidly, causing an almost immediate expansion. Assuming that the concentration of free sodium hydroxide in the pores is low when the reaction has ceased, the gel characteristics of the isotherm, the increase in surface area, and the swelling pressures produced in the pores of the Vycor glass are excellent verification of the Powers-Steinour postulation as related to the physical properties of the alkali-silica com-

plex. The ease of leaching of the alkali-silica complex from within the pores of the glass is apparent from the sorption results of the leached sample. This leaching effect was not evident in the case of the alkali-treated rock.

The plot on Fig. 11, weight gain to expansion, is of considerable interest. The curve for the untreated sample (24- to 30-ft green) is typical of a porous material[12] as is that for the alkali-treated rock except for the latter portion of the desorption path AB, which instead of rejoining the adsorption path, intersects it twice. This illustrates the secondary hysteresis observed in both the sorption and expansion isotherms. This curve suggests that the increase of sorbed water due to alkali treatment of the rock may be due to adsorbed water and the formation of a gel.

There is considerable evidence that the material causing expansion and identified by the characteristics in the isotherms of the treated limestone, whether it be a gel or a hydrate, is produced either as the result of or in association with the dedolomitization reaction. Work to study this relation is being pursued.

CONCLUSIONS

The sorption and expansion isotherms of the alkali-treated limestone establish the presence within the pores of trace amounts of a material that causes expansion when water is made available to it. The mechanism of expansion is similar to that attributed to the alkali-silica complex formed in the pores of Vycor glass although the composition of the material in the two cases may be different. The high sensitivity of this technique for detecting materials which have affinity for water enabled its use in establishing the presence of such a material in limestone having a low pore-volume.

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Características de los isotermas de la absorción y expansión del agregado de la caliza reactiva

Diferencias características se han revelado en los isotermas de la absorción y expansión del agregado de la caliza reactiva, tratado con álcali y no tratado; estos resultados se comparan con los que se obtienen del vidrio Vycor bajo condiciones parecidas. Se concluye que la evidencia establece la presencia, dentro de los poros del agregado, de vestigios de un material que ocasiona la expansión cuando disponedel agua y que el mecanismo de expansión se parece al complejo de álcali-sílice formado en los poros del vidrio Vycor aunque la composicion de los materiales en los dos casos sea diferente.

Caractères des isothermes de sorption et de dilatation de l'agrégat à chaux réactive

On a décelé des différences caracteristiques dans les isothermes de sorption et de dilatation de l'agrégat à chaux réactive, traitée ou non par l'alcali; ces résults sont comparés à ceux obtenus à partir du verre Vycor sous des conditions analogues. On en conclue que l'évidence établit la présence, dans les pores de l'agrégat, de traces d'une matière produisant la dilatation quand l'eau y est disponible, et que le mécanisme de dilatation est analoque au complexe alcali-silice créé dans les pores du verre Vycor, bien que la composition des matières puisse être différent dans les deux cas.

Kenndaten der Sorptions—und Ausdehnungsisothermen reaktiver Kalkstein—zuschlagstoffe

Charakteristische Unterschiede wurden in den Sorptions- und Ausdehnungsisothermen von mit Alkali behandelten und umbehandelten reaktiven Kalkstoff-Zuschlagstoffen gefunden; es wird der Schluss gezogen, dass der Beweis fuer die Anwesenheit von Spuren eines Materials, das die Ausdehnung verursacht, wenn Wasser zur Verfuegung steht, in den Poren des Zuschlagstoffes erbracht wurde, und dass der Ausdehnungsmechanismus aehnlich dem Alkali-Silizium Komplex ist das in den Poren von Vycor Glas gebildet wird, obwohl di Zusammensetzung in den beiden Faellen verschieden sein kann.

Title No. 58-10

Plastic Strain in Prestressed Concrete Beams Under Sustained Load

By JOHN N. CERNICA and M. JEAN CHARIGNON

Fourteen concrete beams were loaded under sustained load at the third-points for 1 year. The beams were identical except for the type of coarse aggregate. All 14 were loaded with a load equal to 45 percent of the average ultimate of six beams tested under stort-time static load. Strain was measured at the center of the beam (a section of pure bending) at four different depths of the beam, viz., top, upper third, lower third, and bottom. The results show that under sustained loading the time-rate of plastic compressive strain in the concrete is initially rapid and then gradually decreases in both the slag and limestone beams. Furthermore, at the end of 1 year, the plastic compressive strain was a little over twice the elastic strain as measured by SR-4 strain gages.

■ CONCRETE IS SUBJECT TO PROGRESSIVE and cumulative deformation under load at common temperatures. Such deformations may cause a substantial reduction of prestress in pretension or post-tension members. It follows, therefore, that in the design of such structures it is necessary to take into account the effect of such deformation. For purposes of discussion, the time-yield of concrete under prolonged constant load will be referred to as "plastic strain." Likewise, the strain obtained at the first instance of loading shall be termed "elastic strain."

A few investigations have been made of the plastic deformation in both cylinders and beams.[1] The information presently available is hardly adequate to take care of the need. It was the purpose of this investigation to contribute further to this field, i.e., to develop a better understanding of the plastic strain in prestressed concrete beams.

Applying the information from cylinder tests to the design of beams, however, may be difficult and possibly even misleading because of the complex relationship between cylinder strengths and beam strengths. To exclude such possibility, prestressed beams were chosen as the test specimen.

TESTING PROGRAM

Specimen

The beam specimens used for this study were of rectangular cross section 5 in. wide, 7.5 in. deep and 8.5 ft long, prestressed with two ⅜ in. wire strands whose centroids were 5.13 in. down from the top (compression face) of the beam.

The concrete was mixed by a 1 cu ft, motor-driven mixer. The cement : fine aggregate : coarse aggregate ratio by weight was 1:2:2.5, with 5.6 gal. of water per sack of cement. The slump was 2.5 in., and the maximum size of the aggregate was ¾ in. The 6 x 12-in. cylinder strength was 5975 psi at 14 days, and 6200 psi at 28 days. The beams and cylinders were steam cured at a temperature of 90 F for 24 hr. The cement was high-early-strength portland cement.

To prestress the beam, a "frame" was designed and built. One end of each prestressing cable was anchored to the frame using a strand vice, the other end was attached to a calibrated load yoke using a strand vice in a turnbuckle. The tension in the cables was applied by tightening a nut on the load yokes.

SR-4 strain gages were installed on a reduced-area section of the load yokes. The yokes were then calibrated in a standard testing machine.

After the desired tension in the beam was obtained, the concrete was placed and cured. The cables were released after curing, thus completing the prestressed beams.

A method was devised by which all 14 beams could be subjected to equal sustained load simultaneously. This was accomplished by arranging them such that the load on one was the reaction on the other, which in turn would react on the following beams. This is shown by Fig. 1, and schematically in Fig. 2.

The load was applied by two tie rods, Fig. 3. On these rods were mounted SR-4 strain gages for determining the sustained load. By calibrating the rods, it was possible to maintain a reasonably constant load by periodically tightening them as loss of load occured due to plastic deflection of the beams. Furthermore, a heavy spring, with a low

Fig. 1 — Prestressed beams being tested under sustained load

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M. Jean Charignon is professor and dean of Youngstown University's School of Engineering. Dr. Charignon obtained his PhD in mechanical engineering at the University of Pittsburgh. He was employed by the Republic Steel Corp. for 13 years as a combustion engineer, and as a power engineer.

constant (1100 lb per in.), was used to minimize the loss of load due to plastic deflection of the beams.

As a check on the load, the spring was calibrated before loading. Thus, by measuring the deformation of the spring, the load could be determinined and checked against that in the tie rods. The deformation of the spring was measured by inside micrometers to less than 0.01 in.

To minimize the effect of friction between the beams and the supporting strips, the beams were placed on rollers.

Strain Measurement

Strain was measured in the center of the span at four depths of the beam: the compression face, one-third of the depth down from the compression face, one-third of the depth up from the tension face, and the tension face. SR-4 strain gages, 6 in. long (Type A-9) were installed on the beams at the four depths. Two gages were installed at each depth and connected in series to give an average reading of the strain. A "strain indicator" was used to read the strain.

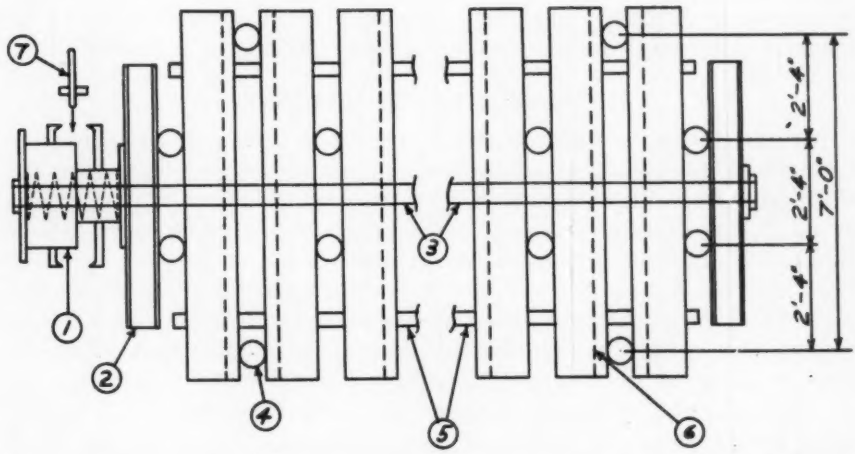

Fig. 2—Diagram of loading apparatus, 1—spring loading device, 2—I-beam, 3—tensioning rods, 4—simple support, 5—beam support, 6—No. 7 wire strand, 7—inside micrometer

Fig. 3—View of strain gages, loading rods, and spring casing

A specially-built switchbox made the reading operation quick and easy, and greatly reduced possible errors due to contact resistance.

To check the plastic strain obtained by the gages, the plastic deflection of the beams was measured with inside micrometers. Watch glasses were pasted on the bottom face of the beams at seven points along the span. The deflection of any two adjacent beams was measured at these points. An analysis of the deflection-time and of the strain-time relationships showed the gages reliable.

Long gages were believed more reliable than short ones. Small cracks in the concrete (such as on the tension face) could cause a large change in the strain measured by short gages. With a long gage, however, the crack is "averaged" over the longer length resulting in a smaller error.

SUMMARY AND CONCLUSIONS

The test results are shown plotted in Fig. 4 and 5. The compressive strain was designated plus, the tensile strain minus. They are the average results of seven beams having limestone and seven having slag as the coarse aggregate. A comparison of the two figures shows an insignificant difference in the behavior and results of the two types of beams. This might suggest that the two coarse aggregates (limestone and slag) did not change the plastic flow characteristics of the two types of beams. This compares interestingly with other studies that show the mineral character of the aggregate to have an appreciable effect on creep or plastic flow.[1] The plastic strain on the compression face of the limestone beams was about 1.07×10^{-3} in. per in., at the end of 1 year of sustained load. For the same period, the slag beams show a plastic strain on the compressive face of 1.2×10^{-3} in. per in., a percentage margin over the limestone beams of 13 percent. The plastic strain 2.6

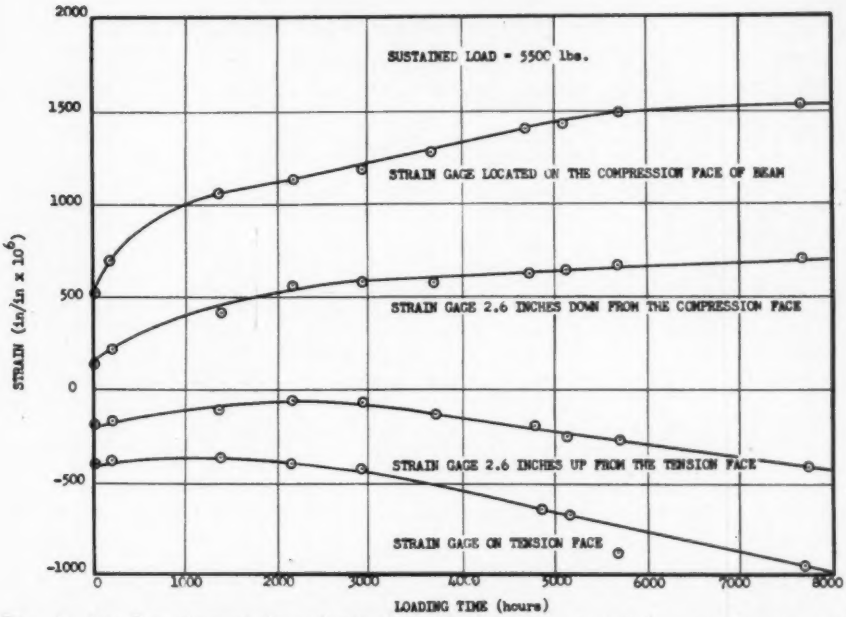

Fig. 4—Loading-time-strain relationship of the "limestone" beams. Average of seven beams

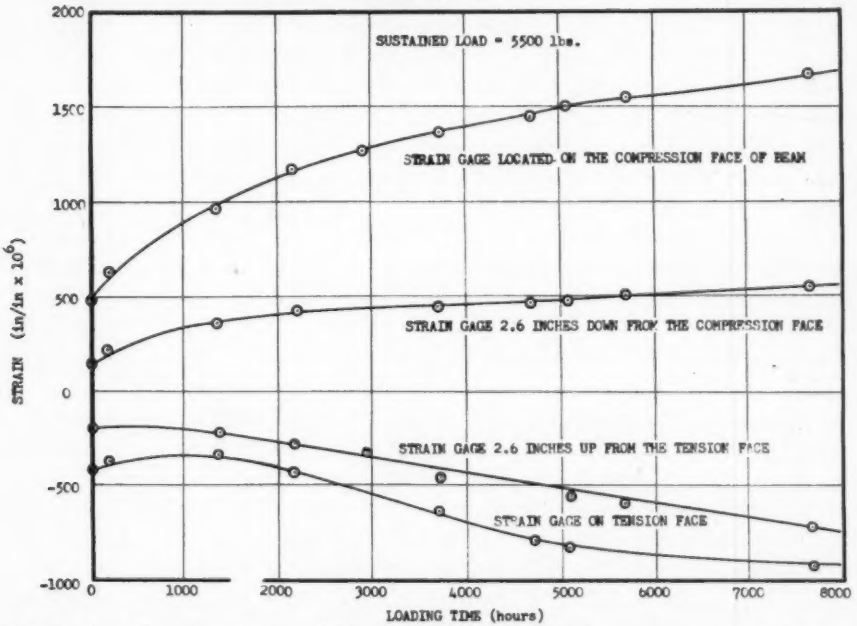

Fig. 5—Loading-time-strain relationship of the "slag" beams. Average of seven beams

TABLE 1—UNIT STRESS IN MATERIALS

Item	Unit stress, psi		
	Ultimate	Yield	Working
Concrete	6200 at 28 days		2790 (45 percent of ultimate)
Wire strand, ⅜ in. diameter	250,000	225,000	137,500 (55 percent of ultimate)

TABLE 2—AVERAGE STRAINS DUE TO PRESTRESSING

Beams	Average strain, microin. per in.	
	Top of beam	Bottom of beam
Limestone	−37*	+375*
Slag	−24	+389

* − indicates compression, + indicates tension

in. down from the compressive faces of the beams was 0.530×10^{-3} in. per in. for the limestone beams and, 0.370×10^{-3} in. per in. for the slag beams, a percentage margin of 30 percent less than the limestone beams.

The strains in the tension portion of the beams seem to behave somewhat curiously. For both types of beams, the tensile strain seems to decrease at first, then gradually increase at almost a constant rate. This might indicate a plastic deformation of the concrete, or concrete paste, in the region responsible for bond, and/or plastic deformation of the reinforcing cable. The latter is not too likely since the stress in the cable was well below its elastic limit. Whatever the reason, the tests indicate that after 1 year of sustained load, the plastic tensile strain exceeds the elastic strain for that load. Note that the net tensile strain is the difference of the tensile strain caused by transverse loading and the compressive strain caused by prestressing (see Table 2). An analysis shows that this difference is almost negligible at first but becomes quite significant with time of loading. This might be of some significance qualitatively if not quantitatively. It might serve to indicate, at least, that cracks are a possibility.

The results of the compressive strains are shown by the two upper curves in Fig. 4 and 5. They show: (1) The time-rate of increase in the plastic strain is initially rapid and then gradually decreases to almost nothing after 1 year of sustained load. Approximately one-third of the total compressive plastic strain over 1 year took place in the first month; approximately one-half of the total occurred during the first one-sixth of the total load time. This follows the tendencies reported for both cylinder and beam tests.[1-11] (2) The plastic strain is over twice the elastic strain as measured by SR-4 strain gages. Both of these conclusions hold for both limestone and slag beams.

ACKNOWLEDGMENTS

The beams and instrumentation were designed and built in the spring of 1958 in the civil engineering laboratories at Youngstown University.

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Deformación plástica en las vigas prefatigadas de concreto bajo carga persistente

Catorce vigas de concreto fueron cargadas bajo una carga persistente a los terceros puntos durante un año. Las vigas eran idénticas con excepción del tipo de agregado grueso. Todas las catorce vigas fueron cargadas con una carga

equal a 45 por ciento del valor definitivo en un término medio de seis vigas ensayadas bajo carga estática de corta duración. La deformación fué medida al centro de la viga (una sección de flexión simple) a cuatro profundidades diferentes de la viga, a saber, la parte superior, la tercera parte superior, la tercera parte inferior, y el fondo. Los resultados muestran que bajo una carga persistente la tarifa horaria de la deformación a la compresión plástica en el concreto es inicialmente rápida y luego disminuye gradualmente tanto en las vigas de escorias como en las vigas de caliza. Además, al fin de 1 año, la deformación a la compresión plástica era un poco más de dos veces más grande que la deformación elástica como medida por los medidores de deformación SR-4.

La déformation plastique de poutres en béton précontraint sous charge continue

On a chargé en continu, quatorze poutres de béton aux troisièmes points pour un an. Les poutres étaient identiques sauf pour la nature de l'agrégat gros. Toutes les quatorze recevaient une charge équivalente à 45 pourcent de la moyenne définitive de six poutres épreuvées sous charge statique à courte durée. La déformation était déterminée au centre de la poutre (une section de flexion pure) à quatre points différents sur la poutre, à savior: le haut, le tiers supérieur, le tiers inférieur, et le bas. Les résultats montrent que sous une charge soutenue, le taux de la déformation plastique à compression avec le temps, dans le béton, est initialement rapide puis il décroît progressivement, chez les poutres tant de cendres que de chaux. D'ailleures, au bout d'un an la déformation plastique à compression était un peu plus que deux fois la déformation élastique, déterminée par les calibres SR-4 de déformation.

Bleibende Formaenderung in vorgespannten Betonbalken unter Dauerlast

14 Betonbalken wurden unter Dauerlast an der dritten Stelle ein Jahr lang belastet. Die Balken waren identisch, mit Ausnahme der Art der groben Zuschlagstoffe. Alle 14 Balken wurden mit einer Last von 45 Prozent der aeussersten Durchschnittslast von sechs Balken unter kurzfristigerstatischer Last belastet. Die Formaenderung wurde im Mittelpunkt des Balkens (ein Profil mit reiner Biegung) an verschiedenen Tiefen des Balkens gemessen, naemlich: Spitze, oberes Drittel, unteres Drittel, Ende. Die Ergebnisse zeigen, dass die Zeitrate der bleibenden Formaenderung unter Druck in Beton unter Dauerlast urspruenglich rasch und dann allmaehlich sowohl in Schlacken als auch in Kalkstein-Balken abnimmt. Ausserdem ist die bleibende Formaenderung unter Druck nach einem Jahr etwas mehr als das Doppelte der elastischen Formaenderung, wie sie mit den SR-4 Spannungsmessern gemessen wurde.

Title No. 58-11

Tests of Rigid Frame Bridge Model to Ultimate Load*

By D. H. PLETTA, ARPAD A. PAP, and CHING-SHENG WU

A one-tenth scale model of an existing skewed reinforced concrete rigid frame bridge was constructed and tested both within the elastic range and up to initial cracking in the concrete and then to ultimate load. The model was fabricated of reinforced concrete using 1/8 in. deformed reinforcing. A total of 57 SR-4 rosette gages were used to measure strains on the surface of the concrete at selected positions. Additional 108 SR-4 gages were fastened to the steel reinforcing. Deflections were measured at 11 points of the deck, and horizontal reactions determined by suitable dynamometers.

Part of the strain data was converted to unit stress for several loadings. Concentrated loadings located at or near the center of the deck produced large local strains, especially in tension. The model was essentially elastic up to about one-third ultimate load and failed in the abutment at the knee.

■ SKEWED RIGID FRAME CONCRETE BRIDGES have been in service for almost a half-century. The design of these folded plate structures, though obviously safe and no doubt conservative was, nevertheless, based on simplifying assumptions that obscured their precise behavior. "Exact" analytical treatments based on the mathematical theory of elasticity seem hopelessly complex for the composite material of which they are made. Analytical approaches based on plate theory have, however, shown considerable progress,[1-4] but have not yet been carried far enough to develop suitable design charts. Analytical approaches based on rigid frame concepts have, of course, been available for some time.[5,6]

The actual behavior of structures is most easily studied in laboratories, but space requirements preclude tests on full scale prototypes. The scale ratio of the model and material of which it is made must be carefully chosen if even static similitude is to be satisfied. Theoretically, a loaded model and its prototype will behave in similar fashion throughout the elastic and inelastic ranges if the two materials are identical and if the concentrated loads vary as the square of the geometric scale.[7] The unit stress in the concrete at ultimate load will, however, vary with the size of the specimen.[8,9] Even so, excellent correlation of model

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and prototype behavior can be achieved with scale ratios as low as 1/8 or 1/10.[9-11]

The test described in this paper was undertaken to determine, as precisely as possible, the variation of strain and deflection with load for a 1/10 scale model of an existing skewed-slab rigid-frame concrete bridge.[12] The entire data, comprising over 15,000 readings cannot, because of space requirements, be reported in detail, but it is available.[13,14] Instead, the strain data was converted to stress at selected sections for three different loading configurations, and two different loads, one before and one after the first crack was observed. Deflections of the deck are presented up to ultimate load.

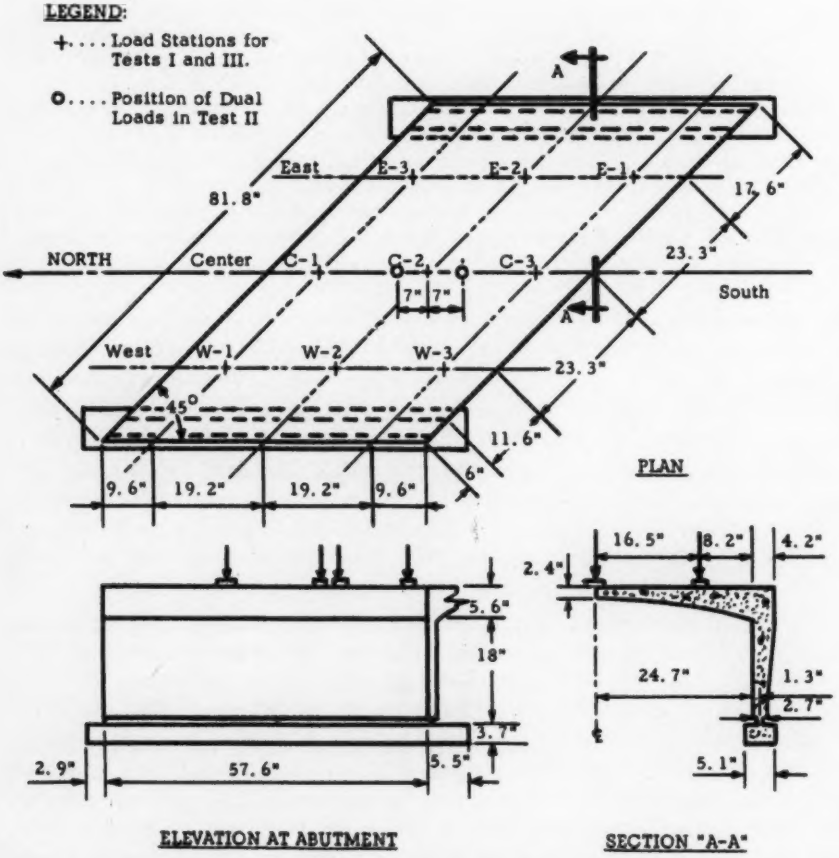

Fig. 1—Diagram of the skewed rigid frame bridge model showing location of load stations

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THE PROTOTYPE AND MODEL

The design of the prototype was described by Brumer.[12] It was constructed on the Palisade Interstate Parkway, Bergen County, N.J., about 1949. AASHO-H20-S16 design loadings were used for both a straight line and plastic theory methods.

The model is a one-tenth scale version of the prototype. The model reinforcing consisted of 1/8 in. diameter deformed wire except for the vertical steel on the inside of the abutments where 1/16 in. diameter smooth rusted wire was used. Its behavior had been checked previously.[11] The model bar spacing was computed so as to keep the steel ratio per unit area identical with that of the prototype. Fig. 1 to 4 show the model dimensions, the location of the load positions, and the arrangement of the reinforcing.

Each bridge footing was cast in a steel box mounted on ball bearings and attached to three dynamometers for measuring the horizontal reactions. A typical pseudo hinge connected the footing to the abutment (Fig. 4, 5). Although dynamometer readings were obtained for every load, subsequent checks of the static equilibrium of half of the bridge failed to yield good agreement between these measured values and

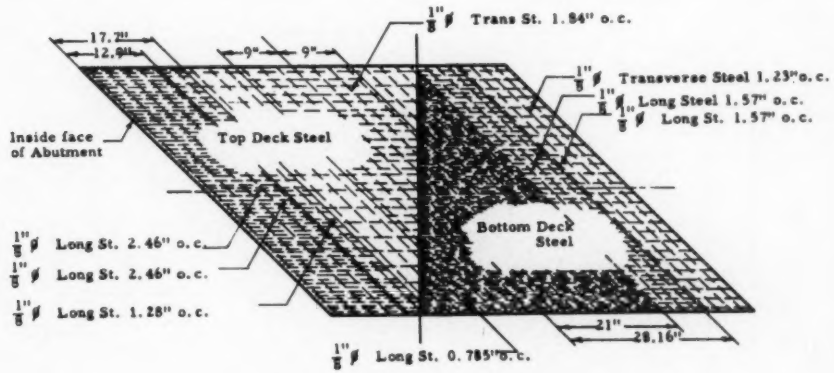

Fig. 2—Arrangement of deck reinforcement

those calculated by integrating the unit stresses along the bridge center line parallel to the abutment. No doubt the inability to locate the precise position of the neutral surface of the deck, especially for the model loaded up to the first visible crack, and possible friction in the footing bearings contributed to the variation of these reactions. Hence, the dynamometer data are not recorded here.

The concrete used in the model was a 1:3.8:0 mix by weight, and contained 6.89 sacks per cu yd of high early strength cement and 8.7 gal. of water per sack of cement. The compressive strength, modulus of elasticity, and Poisson's ratio of this concrete was determined from tests of 2 x 4-in. cylinders and these values at various ages are shown in Table 1. The approximate age of the concrete during the test series

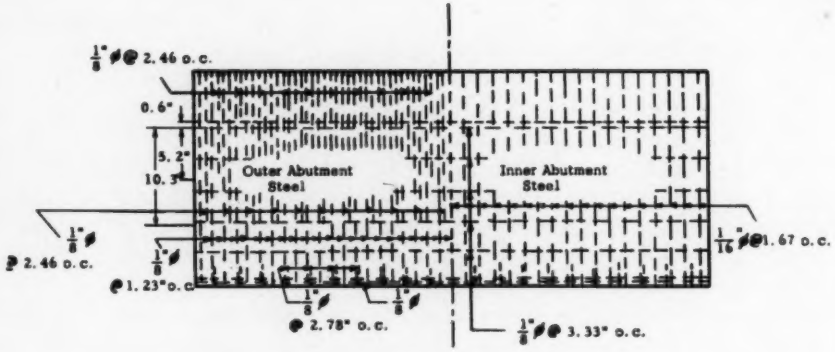

Fig. 3—Reinforcement of model abutment

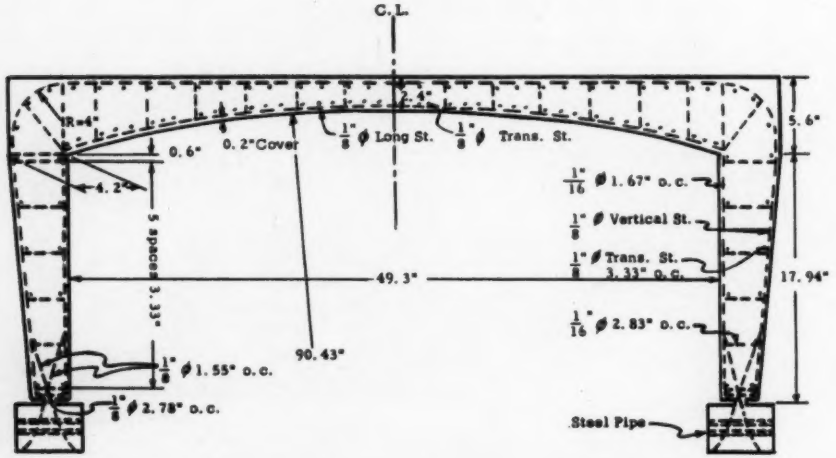

Fig. 4—Model cross section perpendicular to abutments

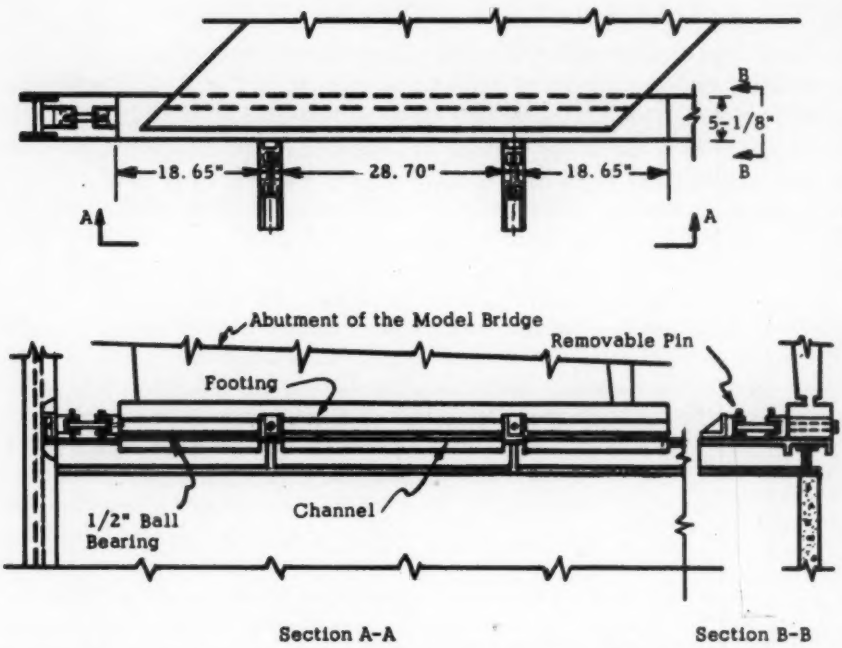

Fig. 5—Diagram of the pseudo hinge connecting the footing to the abutment

was 2-3 years, Test I; 3.4 years, Test II-A; 3.5 years, Test II-B; and 3.8 years, Test III (ultimate). Although the model tests were extended over a considerable period beyond the last test of the cylinders, it was felt that the properties did not change more than a few percent because they were essentially constant after the first few weeks. Values of $E = 4{,}000{,}000$ psi and $\nu = 0.194$ were used in all stress-strain conversions reported later. The cylinders were made from six different batches used for the casting of the model, and were tested in a predetermined order based on statistical theory.

The properties of the 1/8 in. diameter deformed wire, as annealed after the serrations had been rolled into the wire, are as follows:

Modulus of Elasticity (E)	=	29,050,000 psi
Ultimate compressive strength (σ_u)	=	63,900 psi
Johnson's elastic limit (σ_e)	=	39.200 psi
Proportional limit (σ_p)	=	24,800 psi
Yield strength at 0.1 percent offset strain (σ_y)	=	51,000 psi
Percent elongation in 2 in.	=	18
Percent reduction in area	=	76

STRAIN AND DEFLECTION MEASUREMENTS

Three types of bonded wire strain gages were used. Type AR-7 SR-4 rosette gages were applied on the outer surface of the completed model at the intersections of the center lines shown in Fig. 6. Similar gages were applied on the inner surface at identical locations except that one single rosette gage (CID-3) was used on the inner surface at the center of the deck instead of the four gages shown on the outer surface (COD-3N, 3E, 3W, 3S). These four latter gages had to be located away from the center to provide room for the central load. Type A-7 and A-7-4 SR-4 gages were attached to the 1/8 in. and 1/16 in. reinforcement, respectively, before the concrete was placed. They were placed at nearly the same locations as the rosette gages. Although the gages on the steel were waterproofed with several layers of adhesive and then wrapped with rubber tape, a number of them were later found to be grounded to the steel bars. Nevertheless, they were useful as long as a separate neutral wire was used for each gage instead of the usual common ground. The reinforcement, being encased in concrete, was not connected to ground. Transverse (x-direction) gages on the steel bars were given even numbers and longitudinal gages in the θ-direction of the deck or z-direction of the abutments were given odd numbers. No gages were applied in the y-direction of the deck or θ-direction of the abutment since no reinforcement was so placed (Fig. 2, 3).

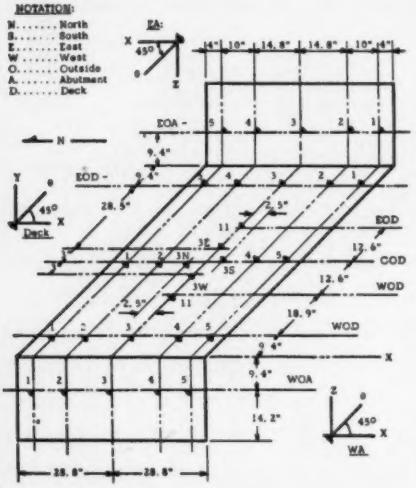

Fig. 6—Location of rosette gages on the outside surface of the model

Deflections of the deck were measured at 11 positions by Ames dials; locations are shown in Fig. 7.

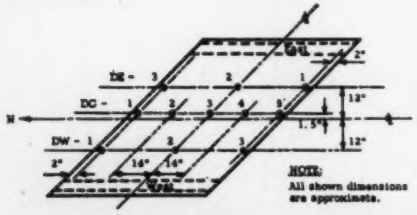

Fig. 7—Location of deflection stations. Notation: D—deflection station; E—east; C—center; W—west; N—north

Strains were measured with a 50 channel Baldwin SR-4 strain scanner and recorder, but all readings were taken using manual rather than automatic operation of this instrument. Since this equipment could accommodate only 50 gages, readings of all 291 gages, including 12 on the dynamometers, required six separate incremental loadings.

Loads were applied in four series. First, for Series I, a 1200-lb load was applied in three increments separately at each of the nine load stations of Fig. 1. The load was applied with a hydraulic jack and measured with a SR-4 load cell. A steel bearing plate 2 in. in diameter and 1 in. thick, placed on a soft rubber cushion, was used to prevent a punching shear failure due to these concentrated loads.

Second, for Series II-A, the load was carried to the first observable crack at 9400 lb in 1500-lb increments. Half of this load was applied at each of two points 7 in. to either side of the center of a north-south center line through bearing plates 1 in. thick by 3.5 in. in diameter (Fig. 1), and was measured with a load cell as before. Strains were read at 50 strategically located gages since this test could be run only once. The crack developed on the under surface of the deck midway between and parallel to the abutments.

Third, for Series II-B, strains in all gages were read in groups of about 50 as the 9400-lb load was reapplied in 1500-lb increments.

Fourth, for Series III, nine calibrated hydraulic jacks (Fig. 8) were used to apply simultaneous loads to ultimate at the nine load stations of Series I (Fig. 1) through 3.5 in. diameter bearing plates. These nine jacks, and a tenth which was placed in a testing machine to measure its load, were connected to a single hydraulic system. The load was applied and measured by the testing machine. All jacks thus applied practically identical loads and this loading simulated a uniformly distributed one. Strains were read on 63 gages including those among the 50 of Series II-A which remained intact. Load increments of 3.6 kips total load (0.4 kips per jack) were applied to the ultimate of 46.8 kips. Failure occurred suddenly in the east abutment (Fig.

TABLE I — TABULATION OF THE PHYSICAL PROPERTIES OF CONCRETE

Age, days	Ultimate strength in compression, psi	Secant modulus at 1350 psi (10^{-6} ×)	Poisson's ratio ν
7†	4,920	3.91	
	4,700	3.20	
	4,650	3.73	0.191
	4,750*	3.62*	
7‡	4,620	3.26	
	4,510	3.88	
	4,540	3.31	0.189
	4,550*	3.48*	
14‡	4,830	3.60	0.194
	5,250	3.77	
	4,150	4.10	
	4,780*	3.82*	
28‡	5,230	4.19	0.225
	6,340	3.88	
	5,140	3.91	
	5,600*	3.99*	
68‡	4,820	3.52	
	4,980	4.09	
	5,300	3.66	0.197
	5,030*	3.76*	
140‡	5,940	6.00§	
	5,770	4.09	
	5,850	3.88	0.194
	5,850*	3.99*	

*Average.
†Molds for 1 day; 100 percent relative humidity, 75 F for 4 days; 50 percent relative humidity, 70 F for 2 days.
‡Molds for 5 days; then open storage in laboratory air (same as model)
§Rejected as erroneous data.

TABLE 2—STRAIN GAGE READINGS FOR THE CONCRETE

Test	Load, lb	Strain, microin.													
		x-direction													
		COD-1	COD-2	COD-3N	COD-3W	WOD-11	WOD-1	WOD-2	WOD-3	WOD-4	WOD-5	WOA-1	WOA-2	WOA-3	WOA-4	WOA-5
I C-2	400	+ 1	+ 1	− 6	− 15	0	− 1	− 1	− 1	− 1	0	0	− 3	− 1	− 1	0
	800	+ 1	0	− 12	− 36	− 2	− 3	− 1	− 3	− 3	− 1	0	− 1	0	0	0
	1200	0	+ 2	− 20	− 52	− 6	− 1	− 3	− 1	− 4	− 2	− 1	− 2	− 2	− 1	− 3
	0	− 8	− 6	− 9	0	− 2	− 2	− 1	− 4	− 1	− 1	+ 1	− 2	+ 1	0	0
II-B	1500	+ 6	+ 5	− 11	− 13	− 9	+ 4	+ 1	0	− 4	+ 2	+ 3	− 1	+ 2	+ 6	− 1
	3000	+ 16	+ 7	− 18	− 21	− 19	+ 5	+ 1	0	− 11	+ 1	+ 4	0	+ 5	+ 4	− 2
	4500	+ 29	+ 7	− 23	− 28	− 29	+ 7	0	− 3	− 20	0	+ 3	− 3	+ 6	+ 2	− 3
	6000	+ 39	+ 12	− 28	− 36	− 38	+ 8	0	− 5	− 26	+ 1	+ 5	− 5	+ 8	+ 8	− 6
	7500	+ 52	+ 19	− 33	− 40	− 46	+ 11	0	− 7	− 30	0	+ 3	− 4	+ 7	+ 15	− 10
	9000	+ 64	+ 22	− 40	− 48	− 55	+ 11	− 4	− 8	− 39	0	+ 4	− 4	+ 11	+ 25	− 8
	9400	+ 66	+ 23	− 41	− 49	− 59	+ 11	− 1	− 11	− 41	− 1	0	− 9	+ 9	+ 29	− 15
	0	− 4	− 6	− 3	− 6	− 2	0	0	− 5	− 3	− 2	+ 5	+ 3	+ 7	+ 17	+ 3
		x-direction													
		CID-1	CID-2	CID-3		WID-11	WID-1	WID-2	WID-3	WID-4	WID-5	WIA-1	WIA-2	WIA-3	WIA-4	WIA-5
I C-2	400	− 2	− 3	+ 68		+ 4	− 2	0	+ 1	+ 1	+ 1	− 3	0	+ 1	− 5	0
	800	− 7	− 7	+132		+ 5	0	+ 1	0	+ 1	+ 2	− 2	+ 1	− 1	− 5	+ 3
	1200	− 9	− 10	+198		+ 8	0	+ 1	0	+ 2	+ 2	0	0	+ 1	− 6	+ 4
	0	− 7	− 6	+ 2		+ 2	+ 1	+ 2	0	0	+ 3	0	+ 1	0	− 1	+ 1
II-B	1500	− 9	− 11	− 18		+ 9	+ 2	+ 2	− 1	+ 3	− 1	+ 6	+ 6	+ 4	0	+ 4
	3000	− 28	− 38	− 62		+ 25	+ 1	0	− 2	+ 3	0	+ 8	+ 5	+ 10	+ 2	+ 7
	4500	− 49	− 64	−104		+ 36	+ 3	+ 3	− 5	+ 6	+ 1	+ 5	+ 5	+ 12	− 2	+ 13
	6000	− 68	− 92	−150		+ 51	+ 4	+ 6	− 7	+ 6	+ 3	+ 8	+ 3	+ 13	− 7	+ 19
	7500	− 88	−122	−200		+ 65	+ 5	+ 9	− 9	+ 8	+ 2	+ 8	+ 5	+ 13	− 10	+ 24
	9000	−113	?	−278		+ 77	+ 6	+ 10	− 11	+ 7	+ 3	+ 10	+ 6	+ 13	− 9	+ 31
	9400	−121	?	−304		+ 82	+ 5	+ 10	− 13	+ 6	+ 3	+ 9	+ 4	+ 11	− 12	+ 32
	0	− 10	?	− 50		+ 6	+ 8	+ 14	+ 1	+ 1	+ 4	+ 5	+ 4	+ 15	+ 7	+ 2
		θ-direction													
		COD-1	COD-2	COD-3N	COD-3W	WOD-11	WOD-1	WOD-2	WOD-3	WOD-4	WOD-5	WOA-1	WOA-2	WOA-3	WOA-4	WOA-5
I C-2	400	− 7	− 11	− 29	− 19	− 1	+ 1	+ 3	+ 4	0	+ 1	+ 2	+ 2	+ 4	+ 1	+ 6
	800	− 14	− 23	− 55	− 38	− 2	+ 3	+ 5	+ 8	+ 2	+ 4	+ 3	+ 3	+ 5	+ 5	+ 15
	1200	− 23	− 38	− 81	− 54	− 5	+ 4	+ 7	+ 11	+ 4	+ 5	+ 3	+ 5	+ 10	+ 7	+ 20
	0	− 9	− 9	− 16	− 10	− 4	− 2	+ 1	0	− 5	− 2	+ 1	0	+ 1	− 1	+ 1
II-B	1500	− 34	− 57	− 74	− 60	− 4	+ 5	+ 15	+ 12	+ 5	+ 15	+ 5	+ 7	+ 16	+ 15	+ 39
	3000	− 74	−124	−158	−128	− 8	+ 15	+ 24	+ 26	+ 18	+ 33	+ 12	+ 14	+ 26	+ 34	+ 86
	4500	−111	−189	−246	−195	− 11	+ 21	+ 32	+ 38	+ 31	+ 51	+ 12	+ 20	+ 45	+ 56	+140
	6000	−150	−252	−333	−260	− 12	+ 31	+ 45	+ 49	+ 41	+ 74	+ 16	+ 27	+ 63	+ 84	+199
	7500	−188	−315	−416	−320	− 12	+ 34	+ 52	+ 62	+ 54	+ 97	+ 17	+ 32	+ 84	+107	+260
	9000	−226	−383	−510	−389	− 13	+ 43	+ 62	+ 72	+ 66	+120	+ 20	+ 37	+109	+144	+337
	9400	−235	−402	−534	−410	− 13	+ 42	+ 67	+ 74	+ 66	+126	+ 19	+ 39	+114	+162	+371
	0	− 8	− 9	− 8	− 15	− 3	+ 3	+ 2	− 3	− 20	+ 1	+ 2	+ 3	+ 14	+ 19	+ 34

		θ-direction														
		CID-1	CID-2	CID-3		WID-11	WID-1	WID-2	WID-3	WID-4	WID-5	WIA-1	WIA-2	WIA-3	WIA-4	WIA-5
I C-2	400	+ 6	0	+ 37		0	− 3	− 3	− 2	− 2	− 1	− 5	− 3	− 7	− 7	− 5
	800	+ 9	0	+ 75		− 1	− 3	− 4	− 7	− 6	− 4	− 4	− 7	− 12	− 13	− 9
	1200	+ 15	− 1	+ 112		− 2	− 6	− 7	− 11	− 11	− 6	− 6	− 10	− 16	− 20	− 13
	0	− 4	− 9	− 4		− 1	+ 1	− 1	0	+ 1	0	− 3	0	0	+ 1	0
II-B	1500	+ 41	+ 23	+ 155		− 2	− 3	− 7	− 13	− 16	− 12	− 5	− 7	− 11	− 14	− 12
	3000	+101	+ 73	+ 385		− 2	− 8	− 16	− 29	− 31	− 23	− 7	− 14	− 27	− 35	− 28
	4500	+166	+133	+ 630		− 8	− 14	− 22	− 47	− 46	− 38	− 12	− 24	− 46	− 54	− 45
	6000	+229	+241	+ 865		− 12	− 17	− 28	− 64	− 64	− 53	− 15	− 32	− 64	− 72	− 61
	7500	+294	+248	+1115		− 19	− 22	− 34	− 81	− 84	− 72	− 17	− 40	− 82	− 89	− 78
	9000	+373	?	+1375		− 26	− 27	− 40	−102	−104	− 93	− 20	− 46	− 99	−109	− 97
	9400	+397	?	+1450		− 30	− 28	− 43	−108	−110	− 98	− 23	− 51	− 99	−116	−102
	0	+ 10	?	0		+ 6	+ 1	+ 8	0	+ 1	+ 6	0	+ 5	+ 6	+ 1	+ 4
		y-direction										z-direction				
		COD-1	COD-2	COD-3N	COD-3W	WOD-11	WOD-1	WOD-2	WOD-3	WOD-4	WOD-5	WOA-1	WOA-2	WOA-3	WOA-4	WOA-5
I C-2	400	− 8	− 18	− 36	− 30	− 5	0	+ 1	+ 2	+ 3	+ 2	0	+ 1	+ 2	+ 3	+ 7
	800	− 17	− 37	− 71	− 55	− 10	+ 1	+ 3	+ 5	+ 7	+ 6	+ 5	+ 5	+ 7	+ 9	+ 18
	1200	− 28	− 60	−103	− 83	− 13	+ 2	+ 4	+ 6	+ 10	+ 9	+ 6	+ 7	+ 12	+ 14	+ 26
	0	− 8	− 10	− 16	− 4	− 4	0	0	− 1	− 2	0	− 1	0	0	0	0
II-B	1500	− 45	− 92	−113	− 93	− 8	+ 8	+ 8	+ 7	+ 19	+ 11	+ 9	+ 12	+ 24	+ 38	+ 65
	3000	− 99	−188	−224	−179	− 13	+ 13	+ 19	+ 18	+ 44	+ 27	+ 18	+ 27	+ 45	+ 79	+138
	4500	−153	−283	−343	−263	− 16	+ 17	+ 27	+ 27	+ 70	+ 41	+ 27	+ 41	+ 74	+134	+224
	6000	−207	−378	−458	−348	− 20	+ 24	+ 36	+ 36	+ 96	+ 60	+ 42	+ 54	+106	+193	+317
	7500	−261	−468	−574	−430	− 23	+ 27	+ 44	+ 47	+124	+ 80	+ 50	+ 66	+141	+256	+414
	9000	−315	−567	−700	−516	− 27	+ 31	+ 53	+ 57	+152	+ 99	+ 59	+ 79	+182	+370	+541
	9400	−331	−593	−730	−544	− 29	+ 38	+ 55	+ 58	+159	+105	+ 59	+ 82	+192	+425	+595
	0	− 15	− 13	− 13	− 22	− 4	+ 7	− 1	− 1	− 1	+ 1	+ 11	+ 6	+ 18	+ 62	+ 58
		y-direction										z-direction				
		CID-1	CID-2	CID-3		WID-11	WID-1	WID-2	WID-3	WID-4	WID-5	WIA-1	WIA-2	WIA-3	WIA-4	WIA-5
I C-2	400	+ 7	+ 11	+ 98		+ 6	− 1	0	− 2	− 4	− 1	− 5	− 2	− 9	− 10	− 8
	800	+ 13	+ 21	+ 194		+ 10	− 1	+ 2	− 3	− 9	− 5	− 5	− 3	− 14	− 20	− 14
	1200	+ 19	+ 32	+ 296		+ 13	− 2	+ 1	− 7	− 13	− 9	− 8	− 8	− 21	− 29	− 22
	0	− 9	− 5	+ 1		− 3	+ 1	0	− 1	+ 1	0	0	0	− 2	− 2	+ 1
II-B	1500	+ 66	+ 88	+ 320		+ 2	0	− 2	− 5	− 15	− 8	− 5	− 6	− 23	− 28	− 25
	3000	+156	+223	+ 830		+ 4	− 5	− 7	− 16	− 30	− 19	− 14	− 17	− 54	− 62	− 54
	4500	+247	+372	+1340		+ 4	− 6	− 14	− 27	− 44	− 33	− 26	− 30	− 89	− 94	− 85
	6000	+339	+521	+1850		+ 5	− 8	− 16	− 38	− 60	− 45	− 34	− 39	−114	−128	−116
	7500	+436	+666	+2360		+ 6	− 11	− 21	− 48	− 79	− 59	− 43	− 50	−146	−162	−150
	9000	+551	?	+2900		+ 4	− 15	− 24	− 61	− 99	− 77	− 54	− 61	−173	−200	−186
	9400	+580	?	+3030		+ 1	− 16	− 27	− 66	−105	− 82	− 58	− 66	−185	−213	−199
	0	+ 17	?	+ 10		+ 4	+ 4	+ 2	+ 1	+ 4	+ 3	+ 3	+ 3	+ 6	− 2	+ 6

TABLE 3—STRAIN GAGE READINGS FOR THE REINFORCEMENT

Test	Load, lb	Strain, microin.													
		x-direction													
		COD-2	COD-4	COD-6	WOD-12	WOD-2	WOD-4	WOD-6	WOD-8	WOD-10	WOA-2	WOA-4	WOA-6	WOA-8	WOA-10
I C-2	400	+ 1	− 5	− 17	+ 1	+ 11	+ 1	+ 3	+ 1	+ 1	+ 9	+ 3	− 1	− 1	− 2
	800	+ 1	0	− 30	+ 3	+ 15	+ 2	+ 3	0	+ 4	+ 11	+ 6	− 1	− 2	− 2
	1200	− 2	− 5	− 46	+ 2	+ 4	+ 3	+ 4	− 2	+ 4	− 4	0	− 2	− 2	− 2
	0	− 4	− 10	− 4	+ 1	0	− 1	0	− 1	+ 2	− 2	− 2	− 3	− 3	− 5
II-B	1500	+ 6	− 6	− 6	− 4	− 1	0	− 1	0	+ 1	0	+ 7	+ 1	+ 2	0
	3000	+ 13	− 14	− 16	− 7	− 3	0	− 4	0	+ 3	− 1	+ 10	− 1	− 3	− 3
	4500	+ 13	− 20	− 19	− 12	+ 2	+ 2	− 10	− 9	− 6	− 1	+ 16	+ 1	− 5	− 4
	6000	+ 24	− 12	− 26	− 18	+ 1	0	− 13	− 12	0	− 2	+ 22	0	− 7	− 6
	7500	+ 20	− 18	− 20	− 23	+ 4	0	− 20	− 14	− 1	− 3	+ 26	− 1	− 11	− 10
	9000	+ 23	− 24	− 25	− 27	+ 8	+ 1	− 27	− 19	0	0	+ 32	− 4	− 16	− 15
	9400	+ 24	− 20	− 26	− 30	+ 10	− 5	− 16	− 19	0	0	+ 34	− 6	− 18	− 17
	0	+ 8	0	− 10	0	+ 3	− 14	− 14	0	− 8	− 2	0	− 2	+ 1	0
		x-direction													
		CID-2	CID-4	CID-6	WID-12	WID-2	WID-4	WID-6	WID-8	WID-10	WIA-2	WIA-4	WIA-6	WIA-8	WIA-10
I C-2	400	+ 1	0	+ 5	0	− 1	Broken	0	+ 1	0	+ 1	0	0	+ 2	+ 2
	800	0	− 1	+ 15	− 1	− 4	"	0	+ 1	+ 1	+ 3	+ 2	+ 1	+ 2	+ 8
	1200	+ 1	− 6	+ 22	− 2	− 10	"	− 2	+ 1	0	+ 2	+ 1	0	+ 2	+ 4
	0	0	− 2	− 1	− 3	− 10	"	− 3	− 2	− 3	− 1	0	− 2	+ 1	0
II-B	1500	− 5	− 4	− 13	+ 3	0	"	− 1	+ 2	+ 1	+ 6	+ 1	+ 2	+ 1	+ 3
	3000	− 15	− 11	− 40	+ 7	− 4	"	− 3	− 1	+ 3	+ 3	0	− 4	− 4	+ 6
	4500	− 26	− 14	− 66	+ 11	0	"	− 7	− 1	0	+ 5	+ 2	− 2	− 3	+ 4
	6000	− 39	− 24	− 98	+ 15	0	"	− 8	0	− 2	+ 7	+ 3	− 2	− 4	+ 7
	7500	− 46	− 29	−124	+ 18	0	"	− 9	− 1	− 3	+ 7	+ 3	− 1	− 4	+ 10
	9000	− 56	− 36	−144	+ 22	− 3	"	− 9	0	− 4	+ 3	0	− 5	− 7	+ 10
	9400	− 60	− 35	−157	+ 23	− 2	"	− 1	+ 1	− 5	+ 4	0	− 4	− 7	+ 10
	0	− 1	− 1	+ 2	0	0	"	− 7	− 6	+ 1	0	− 2	− 7	− 9	− 7
		θ-direction									*z*-direction				
		COD-1	COD-3	COD-5	WOD-11	WOD-1	WOD-3	WOD-5	WOD-7	WOD-9	WOA-1	WOA-3	WOA-5	WOA-7	WOA-9
I C-2	400	− 4	− 3	Broken	0	+ 2	+ 12	+ 4	+ 5	+ 6	− 0	+ 4	+ 4	+ 6	+ 6
	800	− 6	− 11	"	+ 1	+ 4	+ 12	+ 7	+ 9	+ 9	+ 2	+ 3	+ 5	+ 10	+ 10
	1200	− 12	− 18	"	0	+ 4	+ 11	+ 8	+ 11	+ 7	+ 2	+ 3	+ 6	+ 12	+ 15
	0	− 2	+ 2	"	− 2	− 2	+ 1	− 2	− 2	− 1	− 1	− 2	− 7	− 1	+ 2
II-B	1500	− 17	− 28	− 50	+ 1	+ 7	+ 11	+ 12	+ 19	+ 14	+ 10	+ 7	+ 16	+ 20	+ 22
	3000	− 39	− 68	−110	+ 4	+ 13	+ 19	+ 20	+ 35	+ 25	+ 16	+ 6	+ 28	+ 42	+ 47
	4500	− 62	−105	Broken	+ 6	+ 19	+ 32	+ 34	+ 57	+ 36	+ 24	+ 9	+ 36	+ 70	+ 75
	6000	− 83	−143	"	+ 12	+ 24	+ 40	+ 48	+ 78	+ 56	+ 30	+ 12	+ 57	+100	+105
	7500	−102	−175	"	+ 16	+ 30	+ 48	+ 60	+100	+ 73	+ 36	+ 16	+ 73	+129	+136
	9000	−122	−207	"	+ 21	+ 34	+ 58	+ 75	+119	+ 89	+ 42	+ 18	+ 90	+159	+168
	9400	−127	−216	"	+ 19	+ 34	+ 61	+ 77	+120	+ 96	+ 43	+ 17	+ 98	+171	+176
	0	+ 2	+ 5	"	− 1	0	+ 2	− 6	− 8	+ 5	+ 1	− 4	− 2	0	0

		θ-direction							z-direction						
		CID-1	CID-3	CID-5	WID-11	WID-1	WID-3	WID-5	WID-7	WID-9	WIA-1	WIA-3	WIA-5	WIA-7	WIA-9
I C-2	400	8	10	26	0	5	3	0	1	2	3	1	4	1	8
	800	18	26	57	4	6	6	0	4	2	6	3	8	5	15
	1200	16	19	74	7	0	8	10	12	2	2	9	12	10	21
	0	2	4	4	2	1	4	0	0	2	2	3	1	4	0
II-B	1500	47	80	119	6	5	10	14	15	1	5	7	17	16	23
	3000	103	180	254	22	12	21	24	34	15	10	21	42	36	51
	4500	167	268	392	37	22	35	42	52	27	20	33	62	54	84
	6000	225	302	520	50	28	44	57	72	40	22	43	80	71	119
	7500	280	468	648	60	33	54	80	90	50	26	56	102	88	152
	9000	337	572	766	80	36	67	91	108	54	32	66	119	111	180
	9400	342	596	792	83	36	71	96	113	54	32	69	123	112	194
	0	8	6	9	3	4	8	1	4	4	2	19	13	20	18

8) after the deck had deflected over 0.5 in. The outer longitudinal reinforcing actually ruptured at the break in the concrete.

DATA

Only a small part of the total data can be presented here. Tables 2-3 list the strain gage readings for Series I with the 1200-lb load applied at Station C-2 and for Series II-B with the 9400-lb load.

The data for Series I with the 1200-lb load at the other eight stations, for Series II-A, and for Series III had to be omitted here because of the limited space but they are available.[14] Nevertheless, the data presented here are probably the most practical and useful. During Series I, it was found that a 1200-lb load at load stations near the abutments as well as on the north-south center line at C-1 and C-3, produced smaller strains, even near the load itself, than when it was applied at C-2, the center of the deck. Hence, these unlisted strains of Series I are of not too much practical interest, nor are they too significant.

The initial loading to the first visible crack (Series II-A) did produce some nonlinear response of the load versus strain curve, but subsequent reloading (Series II-B) indicated a linear behavior in general. Fig. 9 shows a plot of such data for a few selected gages. Here the strains are plotted in cumulative fashion. Strains for the second cycle are available in Tables 2-3. In Fig. 9 and in Tables 2-3, advantage was taken of load or bridge symmetry and the strains were averaged. For instance, the 1200-lb load of Series I produced a strain in the concrete in the y-direction of -117 microin. at gage COD-2 and -120 microin. at gage COD-4 with the load at C-1 and C-3, respectively. Similarly, a 1200-lb load at C-2 produced a strain on the concrete in the θ-direction of $+25$ microin. at WOA-5 and $+16$ microin. at EOA-5. In the latter illustration the average of $+20$ microin. is recorded in Table 2 for WOA-5.

Fig. 8—Model after rupture showing the arrangement of the nine loading jacks

The linear nature of the load-strain data for the Series II-B loading is rather startling, especially for the large tensile strain on the concrete surface for Gage CID-3 (Fig. 9). This strain of 3030 microin. (in addition to about 1000 microin. of permanent strain, see Fig. 9) is far beyond the supposed tensile ultimate. Other gages subjected to lesser tensile strains behaved in a similar fashion.

Load deflection data were accumulated for each loading series. In general they were considerably more linear than the strain data for the loading of Series II-A, and practically linear for II-B. These data are not reproduced here, but that for the loading to ultimate (Series III) is. Fig. 10 shows that the response was essentially linear to 15,000 lb of total load. Failure occurred at 46,800 lb although the load deflection data were read to only 43,200 lb.

STRESS DISTRIBUTION

The variation of the stress in the concrete over the surface of the deck and abutment was calculated from generalized Hooke's Law equations such as:

$$\sigma_y = \frac{E}{1 - \nu^2} (\varepsilon_y + \nu \varepsilon_x)$$

A value of $E = 4{,}000{,}000$ psi and $\nu = 0.194$ was used. Strains in the steel were converted into stress using $\sigma = E\varepsilon$ with $E = 29{,}050{,}000$ psi.

The stress on the surface of the concrete in the y-direction of the deck and in the z-direction of the abutment due to a 1200-lb load at Positions C-1 and C-2 is shown in Fig. 11 and 12. This stress is plotted on isometric figures perpendicular to the plane of the deck. The abut-

ment surfaces were rotated 90 deg so as to coincide with the x-y plane of the deck. These stresses due to both loadings are roughly of the same order of magnitude except for the tensile values on the inner surface at the center of the deck where a value of +1390 psi exists. Since this exceeds 20 percent of the compressive strength of the concrete, it is possible that the ultimate tension might have been somewhat lower. This possible local cracking is shown graphically on Fig. 12 with dashed lines extending above an arbitrarily selected conservative tensile ultimate of about 600 psi corresponding to an ultimate strain of about 150 microin. Actually, it is possible for mortars such as this to stand ultimate flexural strains of about 240 microin. for a uniaxial state of stress.[15] Nevertheless, no cracking was observed until a 9400-lb load was subsequently applied in Series II-A, and the bridge behaved in a linear fashion during

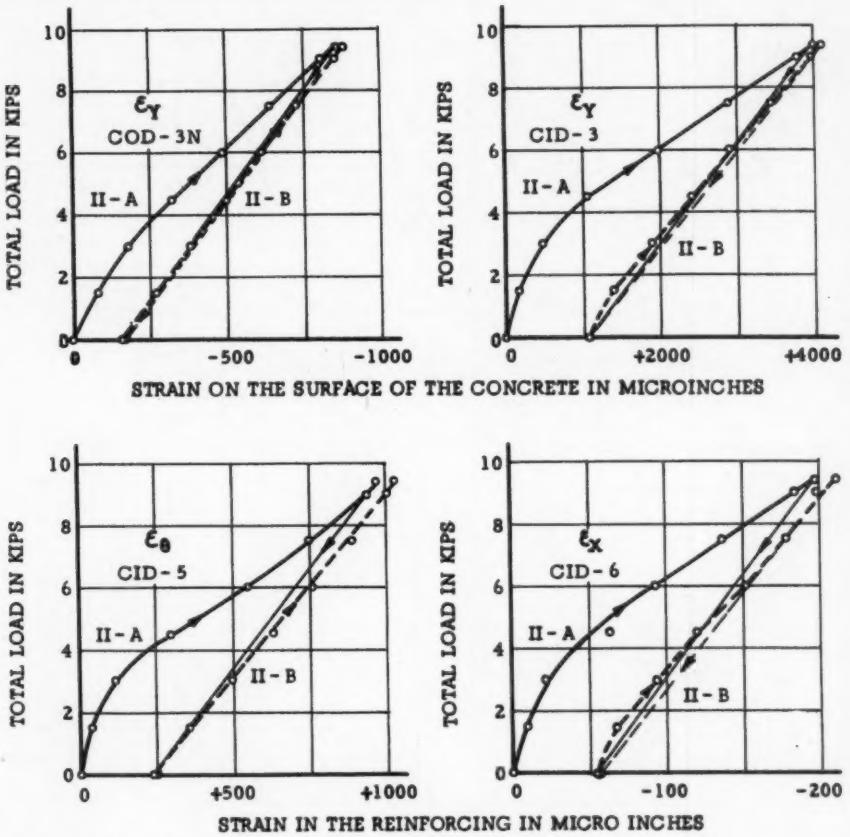

Fig. 9—Cumulative average strain for selected gages due to loading of Series II-A and II-B

the Series I loadings. The fact that the maximum tensile strains for gages nearest the loads appear considerably lower for the load at C-1 than at C-2 is probably due to the fact that no gages were located immediately under C-1 but were located under C-2.

Similarly Fig. 13-16 show the variation in σ_y (or σ_z) and σ_x on the concrete surface, and in σ_θ and σ_x in the reinforcing steel for the 9000-lb load of Series II-B. Maximum stresses of 2341 psi compression in the concrete, and 6010 psi compression and 22,250 psi tension in the steel were calculated. However, it is likely than an even higher value existed for the compression in the steel at the center of the bridge. Gages here

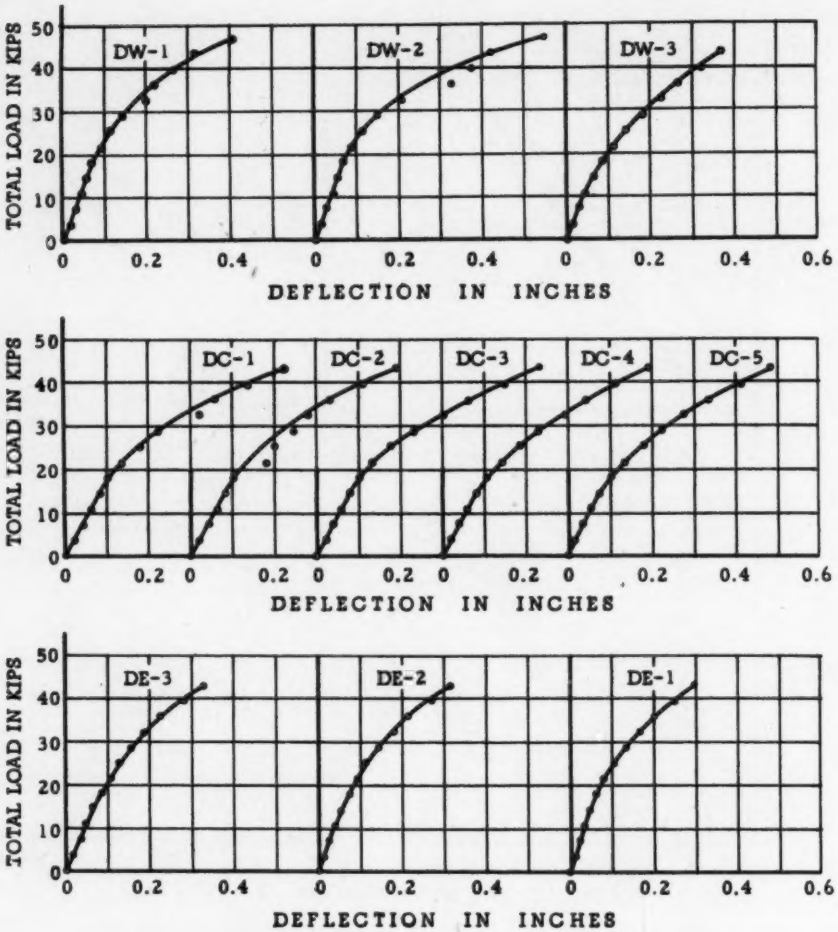

Fig. 10—Load-deflection curves for the ultimate load test

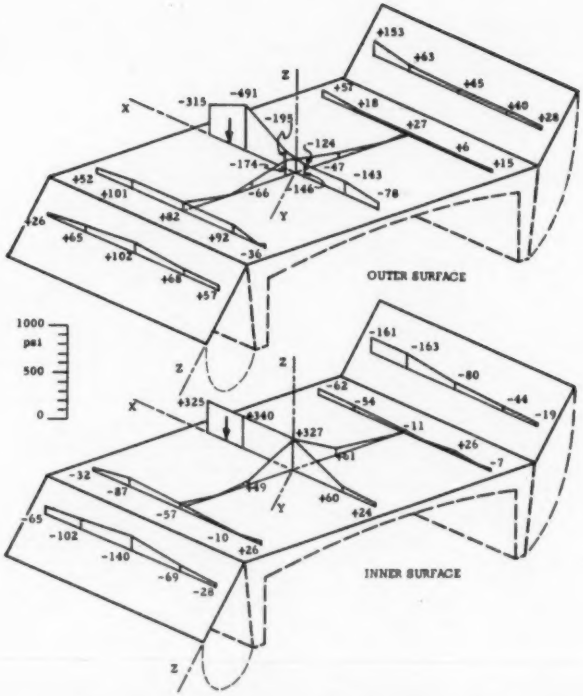

Fig. 11—Stress on the surface of the concrete in the y- and z-directions of the deck and abutment due to a 1200-lb load at C-1

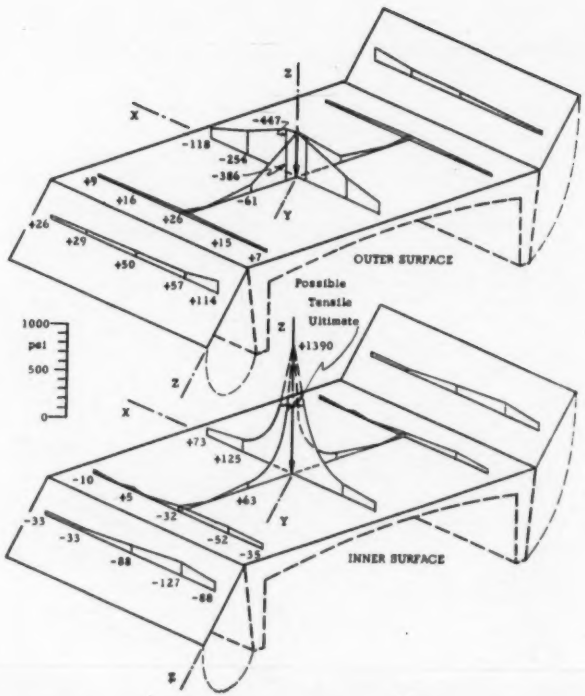

Fig. 12—Stress on the surface of the concrete in the y- and z-directions of the deck and abutment due to a 1200-lb load at C-2

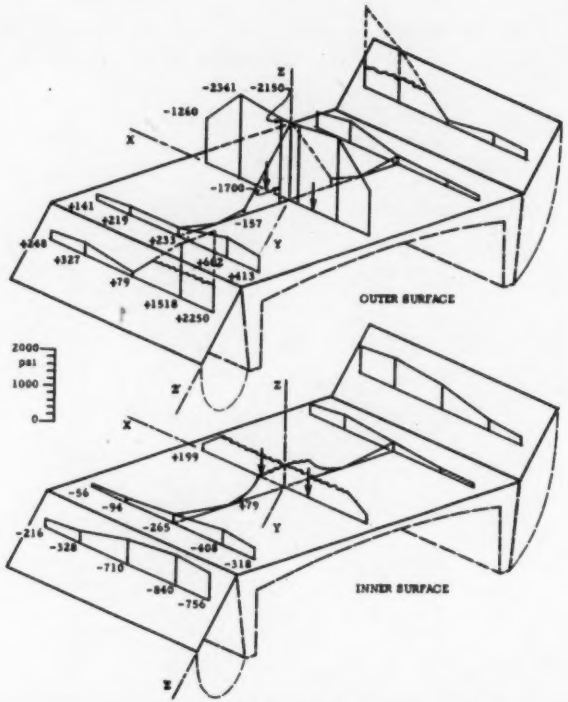

Fig. 13—Stress on the surface of the concrete in the *y*- and *z*-directions of the deck and abutment due to a 9000-lb load of Series II-B

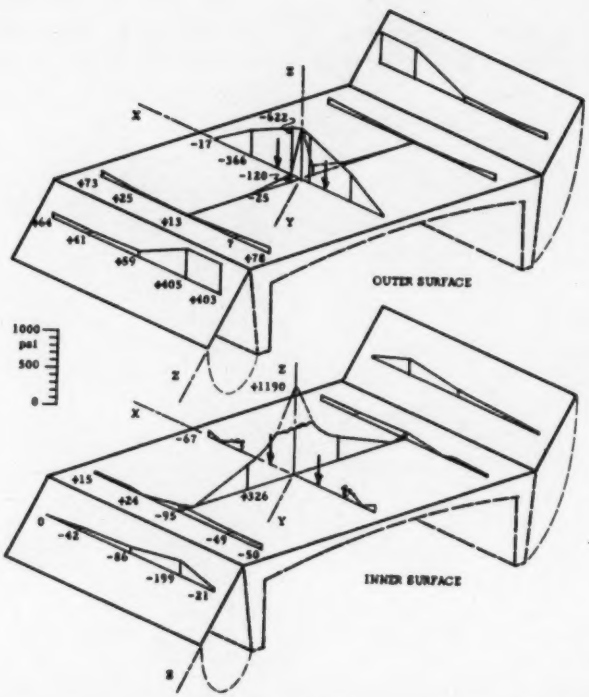

Fig. 14—Stress on the surface of the concrete in the *x*-direction due to a 9000-lb load of Series II-B

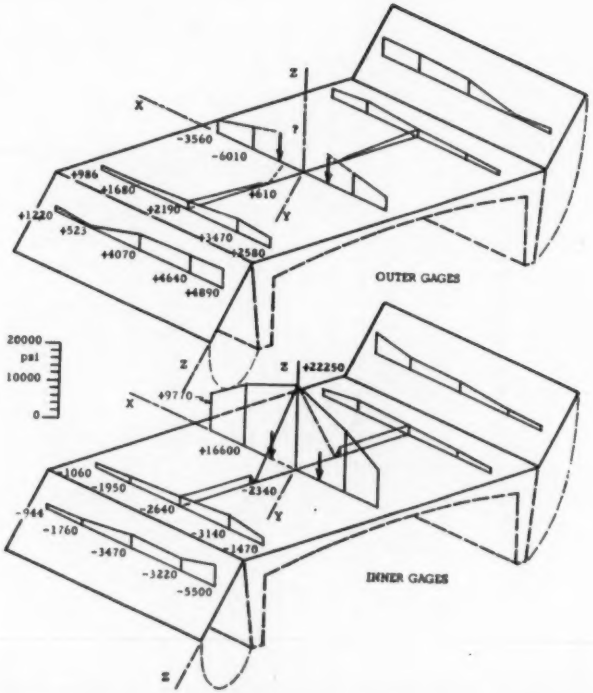

Fig. 15—Stress in the reinforcement in the θ- and z-directions of the deck and abutment due to a 9000-lb load of Series II-B

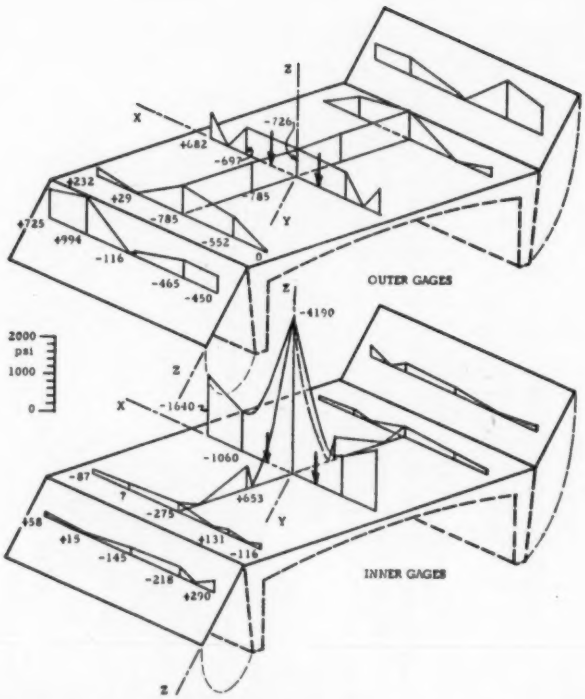

Fig. 16—Stress in the reinforcement in the x-direction due to a 9000-lb load of Series II-B

had ceased to function, so the strain could not be converted to stress. A load of 9000 lb rather than 9400 lb was used here for it is probable that the crack developed somewhat before it was first observed.

CONCLUSIONS

No attempt was made in this report to compare the results with any analytical treatment. It is to be hoped that this will be done. If it is, however, the exact locations of the strain gages attached to the steel should be used if resultant internal moments are to be calculated. The exact locations of these gages are available but are not included here.[14]

If any subsequent tests are made, more suitable reaction dynamometers should be developed. If such data were available, a static check could be made on the resultant internal thrust, moment, and shear on selected cross sections such as the north-south center line of the bridge. Even so, some difficulty will be encountered in deciding the "precise" location of the neutral surface, and of the probable extent of the cracking on the tensile side of the section.

Tests of this bridge and of the skewed slab reported earlier[11] indicate that small scale models, one-eighth to one-tenth the size of the prototype, can be used effectively to determine the behavior of whole structures.

Results reported here justify assumptions of linear response for concrete structures up to about one-third the ultimate load. Such response is, however, not linear during the initial loading up to the first crack, but does exist for subsequent loadings of comparable magnitude.

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Ensayos de un modelo de puente de marco rígido a carga final

Un modelo a escala de 1:10 de un existente puente sesgado de marco rígido de concreto reforzado fué construído y ensayado tanto dentro del límite elástico como hasta la rotura inicial del concreto y entonces a carga final. El modelo fué fabricado del concreto reforzado, empleando una armadura deformada de 1/8 pulgada. Una totalidad de 57 medidores SR-4 de roseta se emplearon para

medir las deformaciones en la superficie del concreto a posiciones escogidas. Unos 108 medidores SR-4 adicionales fueron fijados a la armadura de acero. Las deflecciones fueron medidas a 11 puntos del tablero, y las reacciones horizontales fueron determinadas por dinamómetros convenientes.

Una parte de los datos de deformación fué convertida en esfuerzos unitarios para algunas distribuciones de carga. Las distribuciones de cargas concentradas situadas en o cerca del centro del tablero produjeron grandes deformaciones locales, sobre todo en cuanto a la tensión. El modelo fué esencialmente elástico hasta apróximadamente una tercera parte de la carga final y cayó en el estribo al codo.

Essais à la rupture d'un pont à ossature rigide

Un modèle à l'échelle d'un dixième d'un pont biais actuel en béton armé à ossature rigide a été construit et essayé dans les limites élastiques et jusqu'à la fissuration initiale du béton, puis à la rupture. Le modèle était fabriqué en béton armé avec armature déformée de 1/8 pouce. Une somme totale de 24 calibres à rosette SR-4 déterminait les déformations à la surface du béton à des points prédéterminés. 108 calibres SR-4 de plus, étaient attachés à l'armature en acier. Les flexions étaient mesurées à 11 points sur le plancher et les réactions horizontales déterminées par des dynamomètres convenables.

Une partie de la déformation était rapportée à l'effort par unité pour plusieures charges. Des charges concentrées au, ou auprès du centre du plancher produisaient des déformations locales importantes, surtout à la traction. Le modèle était essentiellement élastique jusqu'à un tiers, à peu près, de la charge à la rupture, et il a failli dans la culée, à l'inflexion.

Versuche mit einem Steifrahmenbruecken-Modell bis zur Grenzbelastung

Ein Modell einer existierenden abgeschraegten Eisenbeton-Steifrahmenbruecke wurde in einem Masstab von 1/10 der natuerlichen Groesse gebaut und innerhalb der elastischen Grenze und bis zum Beginn von Rissbildungen in Beton und weiter bis zur Grenzbelastung geprueft. Das Modell wurde aus Eisenbeton mit Spezialeisen in einer Staerke von 1/8 Zoll hergestellt. 57 SR-4 Rosettenmesser wurden zur Messung der Formaenderungen an der Oberflaeche des Beton an vorbestimmten Stellen verwendet. 108 weitere SR-4 Messer wurden an der Stahlbewehrung befestigt. Durchbiegungen wurden an 11 Stellen der Bedielung gemessen und waagrechte Reaktionen wurden durch geeignete Leistungsmesser bestimmt.

Ein Teil der Formaenderungsdaten wurde fuer verschiedene Lasten in spezifische Belastung umgerechnet. Konzentrierte Lasten im oder nahe dem Mittelpunkt der Bedielung erzeugten grosse oertliche Formaenderungen, besonders unter Spannung. Das Modell war besonders elastisch bis zu ungefaehr ein Drittel der Grenzbelastung, und versagte im Widerlager an der Knickstelle.

CONCRETE BRIEFS

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History of Air-Entraining Cements

By W. C. HANSEN*

THE USE OF air-entraining cements to produce concrete that is resistant to deicing salts in conjunction with freezing and thawing solved a serious problem from the standpoint of the durability of concrete pavements but it also was a big stride forward in providing concrete with highly improved workability characteristics.

It appears that cement technologists to some extent have lost sight of the experimental work that led to the development of air-entraining cement and, also, there appears to be some feeling that this development is based on an accidental discovery. It is desirable, therefore, to call attention to the early work in this field.

In 1943, Kennedy[1] made the following statement: "During the years 1932-1935, intensive work was done by the Dewey & Almy Chemical Co., in connection with the application of a combined air-entraining and dispersing agent to portland cement concrete. Cooperative studies made by Professor C. H. Scholer in the early thirties showed that the entrainment of small amounts of air improved the durability of such concrete. Parallel studies were made by this company, which included investigations of the effect of air-entraining agents on other properties of concrete, also the individual effects of (1) the air-entraining agent, and (2) the entrained air. Though the beneficial effects of air-entraining agents on resistance to freezing and thawing were recognized, the accompanying reductions in density and strength were deemed sufficient reason to discontinue work on this subject at the time."

In 1944, Gonnerman[2] stated: "In 1935 and 1936, some exploratory tests were made to determine the effect of a number of wetting agents on the water requirements, plasticity and strength of portland cement mortars. In a mix of fixed proportions, all of the materials tried increased the air content of the mortar. In those exploratory studies, durability tests were not made and it was not until more extensive studies were undertaken subsequently that the beneficial effect of entrained air on resistance to scaling and to freezing and thawing of concrete was definitely established."

In 1940, Moore[3] made the following statement: "Increasing use of salt — either common salt or calcium chloride —to remove ice from concrete roads on grades, curves and intersections has led

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*Member American Concrete Institute, Consulting Chemist, Valparaiso, Ind.

to many complaints of scaling of the concrete surface. Accordingly, three years ago, a study was begun in the laboratory of the Universal Atlas Cement Company to find means of preventing this scale."

In this study, which was carried out during the winter of 1937-1938, small concrete slabs, molded with dikes to form a pond which would hold about ¼ in. of water, were covered with water and exposed out-of-doors. Whenever the water froze solid, the ice was melted by an application of solid flake calcium chloride in the morning and the solution was poured off the slab. The pond was then refilled with water and the slab was exposed again to natural freezing.

Blended and natural cements

According to Moore, Paul[4] reported in 1938 on an experimental road built in 1935-1936 which showed that concrete made with a blend of portland cement and a natural cement was much more resistant to freezing and thawing than the concretes made with normal portland cements. In the tests reported by Moore, a concrete slab made with a blend of this natural cement and portland cement and one made with a cement produced by intergrinding portland cement with small amounts of a fish oil stearate showed high resistance to the treatment with calcium chloride. Moore states: "Since the freshly mixed concrete of the two slabs of greatest resistance contained more entrained air than the other slabs, it was assumed that entrained air had something to do with prevention of scaling. A search was made for other materials which would entrain air; subsequent tests confirmed the inference."

Studies of the natural cement showed that it contained 0.07 percent of fat or grease. Other tests showed that Vinsol resin which was being tested as a grinding aid caused the entrainment of air. It appears that Moore and his associates were the first to recognize that entrained air would prevent the scaling that resulted from the use of deicing salts.

Experimental pavements

The Universal Atlas Cement Co.,* during Oct. 21-27, 1938, constructed an experimental pavement at their Hudson, N. Y., plant, to establish the validity of their laboratory findings. The cements used were two portland cements, a blend of portland and natural cement, a cement ground with 0.05 percent fish oil stearate, and a cement ground with 0.05 percent Vinsol resin. Salting tests made on this pavement clearly established the merit of entrained air in producing concrete that was resistant to deicing salts. The experience in placing this pavement also showed that the entrained air almost eliminated segregation and bleeding of the concrete and produced a substantial increase in the workability characteristics of the concrete.

The State of New York Department of Public Works[5] also in 1938 built an experimental pavement with various portland cements used alone and as blends with natural cements. A cement ground with 0.05 percent beef tallow was also used in this road because beef tallow was regularly ground with the natural cement used in the blends. The results on this test road confirmed the results of the laboratory tests with entrained air.

Anderson[6] gives data for other experimental roads built in 1939 and 1940. One of his conclusions was "The increased durability attained by the use of treated cements seems to be a function of the air entrained in the concrete during mixing."

It is seen from this review that, from about 1930 on, several investigators had observed the effects of certain materials in promoting the entraining of air in concrete. It also appears that

*Now Universal Atlas Cement Division, United States Steel Corp.

Moore and his associates were the first to recognize that entrained air might be the solution to the problem of scaling when deicing salts were used and their experimental road at Hudson, N. Y., was the first field experiment designed primarily to test this hypothesis.

Data[6] were available from which one could have predicted that entrained air would increase the resistance of concrete to freezing and thawing but not necessarily to the effects of deicing salts. Investigators in England had reported in 1932 that the resistance of natural stones to freezing decreased when the absorption of water exceeded about 90 percent of the total absorption of the stone. Since water expands approximately 9 percent during freezing, the natural conclusion was that the pores filled with air but unfilled with water served as space into which the water could expand during freezing and, accordingly, prevented the freezing of water from disrupting the stone. Likewise, one could have concluded that, if this was true for natural stones, it was likely also true for concrete which might be classed as a synthetic stone. These data also showed that the fine pores in stone were much more effective than coarse pores in resisting the effects of freezing and thawing, which has been found to be true for concrete.

Discovery not accidental

One occasionally hears statements to the effect that beneficial effects of entrained air in concrete were discovered accidentally. Such a statement might apply to most discoveries because, in any truly research project, the investigator almost certainly does not anticipate what the results of the study will reveal. In the case of entrained air in concrete, the fact that certain materials caused entrainment of air and the effects produced by this air were observed as results of systematic researches. Hence, the only accidental feature was that the investigators had not realized previously that these results would be observed.

Moore and his associates were working on a project which had for its objective the solution of the scaling problem. Field experience had already demonstrated that a certain natural cement seemed to contain something that imparted scaling resistance to blends of this cement with portland cements. They analyzed this natural cement and found that it contained about 0.07 percent of a saponifiable grease. They also observed that this cement caused the entrainment of air and that fish oil stearate, an impure calcium stearate used in a masonry cement, and Vinsol resin used as a grinding aid in the manufacture of cement also caused the entrainment of air. From this, they concluded that the entrained air was the effective ingredient and began field work that led to the development of air-entraining cements. Hence, it seems inaccurate to class the result of this type of work as an accidental discovery.

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Spacing of Piles for Equal Reaction

By K. R. PATEL*

THE TASK OF DETERMINING the spacing for equal loads on all piles for the foundation of abutments or retaining walls can greatly be facilitated by the equations given here for piles arranged in two, three, and four rows. The method used to determine the pile reaction is commonly used by engineers. The derivations of the equations are simple but laborious and hence are eliminated here to save space. The following two assumptions were made to derive the equations listed herein:

1. The ratio of reaction to settlement is constant

2. The pile cap is rigid so that the plane of the foundation remains plane after deflection

TABLE I—VALUES OF T FOR DIFFERENT VALUES OF A

A	0.00	2.50	3.00	3.50	4.00
T	1.00	0.98	0.97	0.96	0.95

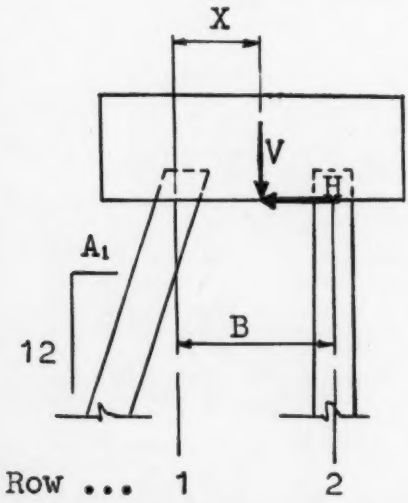

Fig. 1—Spacing piles in two rows

Notation

R = maximum allowable pile reaction

V = vertical component of resultant force per linear ft of wall

H = horizontal component of resultant force per linear ft of wall

$S_1, S_2, \ldots$ = spacings of piles in Rows 1, 2, . . .

$A_1/12, A_2/12, \ldots$ = the pile batter for Rows 1, 2, . . .

X = distance of the resultant load from Row 1

Y = distance between Rows 1 and 2

Z = distance between Rows 1 and 3

B = distance between the front and the rear rows of piles

E = 70 to 80 percent of the total horizontal load H which may be used to design batter piles to approximate the effect of the lateral resistance of the footing and the piles against the soil

$T = 12/\sqrt{(12)^2 + (A)^2}$

SPACING PILES IN TWO ROWS (see Fig. 1)

$$S_1 = \frac{RBT_1}{V(B-X)} \quad \ldots\ldots (1)$$

$$S_2 = \frac{RB}{VX} \quad \ldots\ldots (2)$$

provided

$$H \leq (RT_1A_1)/(12ES_1) \quad \ldots\ldots (3)$$

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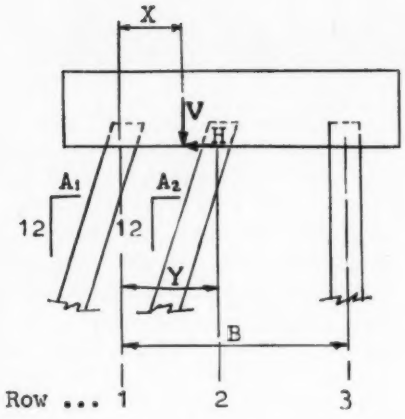

Fig. 2—Spacing piles in three rows

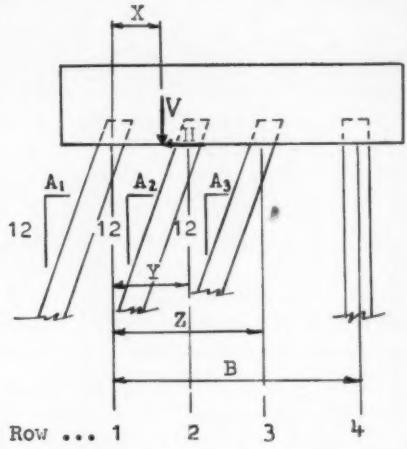

Fig. 3—Spacing piles in four rows

SPACING PILES IN THREE ROWS (see Fig. 2)

$$S_1 = \frac{2R(Y^2 - YB + B^2)T_1}{V(Y^3 - XY - XB + B^2)} \quad \ldots\ldots (4)$$

$$S_2 = \frac{2R(Y^2 - YB + B^2)T_2}{V(B^3 - XB - YB + 2XY)} \quad \ldots\ldots (5)$$

$$S_3 = \frac{2R(Y^2 - YB + B^2)}{V(Y^2 - YB + 2XB - XY)} \quad \ldots\ldots (6)$$

provided

$$H \leq \frac{R}{12E}\left[\frac{T_1A_1}{S_1} + \frac{T_2A_2}{S_2}\right] \quad \ldots\ldots (7)$$

SPACING PILES IN FOUR ROWS (see Fig. 3)

$$S_1 = \frac{R(3Y^2 + 3Z^2 + 3B^2 - 2YZ - 2ZB - 2BY)T_1}{V(Y^2 + Z^2 + B^2 - XY - XZ - XB)} \quad \ldots\ldots (8)$$

$$S_2 = \frac{R(3Y^2 + 3Z^2 + 3B^2 - 2YZ - 2ZB - 2BY)T_2}{V(B^2 + Z^2 - BX - BY + 3XY - YZ - XZ)} \quad \ldots\ldots (9)$$

$$S_3 = \frac{R(3Y^2 + 3Z^2 + 3B^2 - 2YZ - 2ZB - 2BY)T_3}{V(Y^2 + B^2 + 3XZ - XY - BX - BZ - YZ)} \quad \ldots\ldots (10)$$

$$S_4 = \frac{R(3Y^2 + 3Z^2 + 3B^2 - 2YZ - 2ZB - 2BY)}{V(Y^2 + Z^2 + 3BX - BY - XY - BZ - XZ)} \quad \ldots\ldots (11)$$

provided

$$H \leq \frac{R}{12E}\left[\frac{T_1A_1}{S_1} + \frac{T_2A_2}{S_2} + \frac{T_3A_3}{S_3}\right] \quad (12)$$

EXAMPLE (see Fig. 2)

Let R= 60 kips, V = 40 kips per ft, H = 12 kips per ft, X = 2.65 ft, Y = 3.25 ft, B = 9.50 ft; and taking A_1 = A_2 = 3.50, T_1 = T_2 = 0.96. Then from Eq. 4, 5, and 6, we get S_1 = 3.01 ft, S_2 = 3.92 ft, and S_3 = 9.79 ft. Using E = 0.80 in the Eq. 7, we found that $H < 12.33$. Therefore, S_1, S_2, and S_3 are the required spacings of the piles.

CHECK (see Fig. 4)

The vertical reaction per linear ft of wall in each pile row is given by

$$\frac{V}{N} + \frac{VEC}{I}$$

where

N = No. of rows of piles

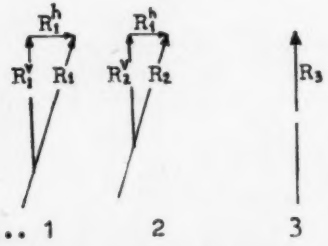

Fig. 4—Vertical reactions of piles in three rows

E = eccentricity of load V with respect to the center of gravity of the rows of piles

I = moment of inertia of pile rows about its' center of gravity

C = distance of the pile row from the center of gravity of pile rows

In the example, N = 3, E = (9.50 + 3.25)/3 − 2.65 = 4.25 − 2.65 = 1.60 ft, and I = $(4.25)^2$ + $(4.25 - 3.25)^2$ + $(4.25 - 9.50)^2$ = 46.625 row ft^2. The values of C_1, C_2, and C_3 are, respectively, 4.25, 1.00 and −4.75 ft for Rows 1, 2, and 3.

Hence, R^v_1 = 13.33 + 5.83 = 19.16 kips per ft, R^v_2 = 13.33 + 1.37 = 14.70 kips per ft, and R_3 = 13.33 − 7.21 = 6.12 kips per ft. Now $R^h_1 + R^h_2 = EH$ = 0.80 × 12 = 9.60 kips per ft. Therefore $A_1 = A_2 = 12(R^h_1 + R^h_2)/(R^v_1 + R^v_2)$ = 3.40 minimum.

Use $A_1 = A_2$ = 3.50, then $T_1 = T_2$ = 0.96

Hence,

R_1 = 19.16/0.96 = 19.96 kips per ft, and S_1 = 60/19.96 = 3.01 ft;

R_2 = 14.70/0.96 = 15.31 kips per ft, and S_2 = 60/15.31 = 3.92 ft;

R_3 = 6.12/1.00 = 6.12 kips per ft, and S_3 = 60/6.12 = 9.80 ft.

These spacings S_1, S_2, and S_3 agree with those computed from Eq. 4, 5, and 6, respectively.

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CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

New bridge at the port of Kassel (Die neue Hafenbrücke in Kassel)

KURT KOZNIETZKI, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 8, Aug. 1960, pp. 171-174
Reviewed by RUDOLPH SZILARD

The 50 year old structural steel bridge over the river Fulda has been replaced by a prestressed concrete bridge, continuous over three spans (66.5, 277, and 66.5 ft). The main girders of the center span have I shapes built together monolithically with the transverse floor beams to achieve three-dimensional action. Box girder sections have been used for the exterior spans. The girders have been post-tensioned by a method similar to Leonhard's prestressing system.

This flyover has structural elegance

Engineering (London), V. 191, No. 4948, Feb. 17, 1961, pp. 246-247
Reviewed by ARON L. MIRSKY

Elevated highway, carrying Great West Road over congested road and rail complex at Hammersmith, in the London area, is a multiple-span, continuous structure of 16 spans; the main structural element is a continuous hollow spine supported on a line of single columns and supporting cantilevers (aptly named "inverted coat hangers") which in turn support the dual roadway. Elements of this structural system are precast, with continuity assured by cast-in-situ joints and longitudinal prestressing. An elaborate system of measurements during and after construction will check design assumptions.

Enseleni Bridge for the Natal Provincial Administration

Public Works of South Africa (Johannesburg), Aug. 1960, pp. 18-19
Reviewed by D. G. NORMAN

Recently completed three-span prestressed road bridge has 192 ft central span, and features in situ cantilever and anchor arms (part box section) with precast "fish-bellied" beams for suspended span. Piers and abutments are on concrete caissons. One 200-ton caisson had to be moved to the river bank while damaged steelwork of an old railway bridge was removed from the bed.

Construction Techniques

Novel column and beam system used in new luxury flats

Construction in Southern Africa (Johannesburg), Nov. 1960, pp. 35-39
Reviewed by D. G. NORMAN

Four story block of 36 flats recently built in Johannesburg uses two heavy cast-in-place beams along sides of building, with precast girders spanning between them. Slab forms were supported on temporary wood sills along the precast girders, which latter were propped. Result was elimination of all propping to cast-in-place slabs.

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For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

Friedrich Street filling station in Stuttgart (Tankstation Friedrichstrasse in Stuttgart)

OTTO JAGER, *Deutsche Bauzeitung* (Stuttgart), V. 65, No. 5, May 1960, pp. 278-282, 284
Reviewed by ARON L. MIRSKY

Clearance and other considerations led to use of "flying wing" canopy, which is supported on four columns arranged in a central core 4.5 x 3.6 m, resulting in overhangs of 11.70 m with a clear central span of around 3.60 m. Canopy slab is heavily reinforced normal to longitudinal "backbone" containing prestressing cables.

Study and improvement of under water concrete placing (Etude et perfectionnement d'une technique de béton immergé)

J. BOUVIER, *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), No. 146, Feb. 1960, pp. 149-180
Reviewed by HENRI PERRIN

The main problems encountered by underwater concreting are given along with a summary of the most usual practices. The most important factor is the capacity for retaining water and some rules are specified in this regard. Three general operating schemes are described and some new devices and processes are also shown. A summarized study of some completed structure closes this exhaustive report.

Reconstruction of the embankments of the Maurienne Valley, France; concreting by frosty weather (Réfection des digues de l'Arc en Maurienne, bétonnage par temps de gel)

C. BARON, *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), No. 144, Dec. 1959, pp. 1349-1368
Reviewed by HENRI PERRIN

Following an exceptional flood, the protective embankments of the Maurienne Valley were seriously damaged and their reconstruction was possible only during the winter, although the job location was at about 3000 ft.

The basic principles followed were (1) to cure the concrete for 48 hr after concreting, since it is not sensitive to frost afterwards, and (2) to assume that after reheating, the concrete hardening will resume though it may have been subjected to frost.

The specifications also called for a low water-cement ratio and addition of an air-entraining plastifier. Various tests conducted to determine the variation of concrete strength as a function of the outer temperature are reported.

Foundation for roll grinding machines

P. H. ALLAWAY, *The Engineer* (London), V. 210, No. 5460, Sept. 16, 1960, pp. 457-458
Reviewed by ARON L. MIRSKY

To provide adequate stiffness and other necessary elements, foundation block was made 7.87 ft deep, in the form of a two-legged "arch." The legs were precast alongside the pit and then lowered into it; the top was then cast and the machine erected, after which the load was transferred to the springs by jacks.

Dams

Heightening gravity dams

Engineering (London), V. 190, No. 4940, Dec. 23, 1960, p. 854
Reviewed by ARON L. MIRSKY

Brief discussion of the Coyne (anchor stressing) method, currently being used for the first time in the United Kingdom for the heightening of Argal Dam for the Falmouth Borough Council.

Aswan hydro-electric scheme

VALTER FURUSKOG and GEOFFREY F. KENNEDY, *Proceedings*, Institution of Civil Engineers (London), V. 17 (Session 1959-60), Oct. 1960, pp. 201-218
Reviewed by ARON L. MIRSKY

Design and construction features (civil, electrical, and mechanical) of the 300 million-watt hydroelectric scheme at Aswan, on the Nile River, a project which will assuredly be noted in the history books, for its socio-political aspects if not for its engineering features.

Two companion papers, in the same issue, cover other aspects of the scheme: "Recent Developments in Nile

Control," by Abdel Aziz Ahmed (pp. 137-180), and "An Analytical Study of the Storage Losses in the Nile Basis, with special reference to Aswan Dam Reservoir and the High Dam Reservoir (Sadd-el-Aali)," by Abdel Aziz Ahmed (pp. 181-200).

Design

Cylindrical northlight shells, design using polynomes (Kreissegment-Shedschalen, Berechnung mit Polynomen)

W. Fuchssteiner, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 9, Sept. 1960, pp. 205
Reviewed by Rudolph Szilard

The unsymmetrical shape of the Northlight shell requires the satisfaction of eight boundary conditions. After neglecting the unimportant internal forces, the stress has been expressed in polynominal form. A numerical example shows the practical applicability of the derived method.

Segmental construction of a northlight shell hall utilizing Dywidag prestressing method (Abschittsweise Herstellung einer Schalen-Shed-Halle unter Anwendung von "Dywidag" Spannverfahrens)

D. Arnold, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 7, July 1960, pp. 145-150
Reviewed by Rudolph Szilard

To satisfy the even illumination requirements of a tire manufacturing company northlight shell halls have been designed and constructed, using Dywidag prestressing methods. The alteration required after a few years of service has changed the structural system of the existing building. The shells, continuous in their longitudinal direction, were originally supported at 80 and 47 ft. After the alteration the span ratio became 1:1, representing a continuous shell with two equal spans. Describes the construction methods used, including the splicing of the prestressing strands. The structural analysis covers the usual load conditions consisting of dead, snow and wind loads.

New Canadian structural test installation

W. R. Schriever and W. G. Plewes, *The Engineering Journal* (Montreal), V. 44, No. 1, Jan. 1961, pp. 35-41
Reviewed by Aron L. Mirsky

Design and construction of facility for testing beams, slabs, columns, roofs, walls, and other structural elements of relatively large size (about 50 ft for columns, 20 x 20 ft for slabs) under large loads (up to 50 tons at each of 42 anchorage points). Reinforced concrete test floor is anchored to underlying bedrock by pretensioned rock anchors. Facility serves Canada's National Research Council's Division of Building Research.

Proof of the safety-factor of prestressed concrete sections having compression zones in plate, triangular, and trapeze forms (Bruchsicherheitsnachweis vorgespannter Betongverschitte mit Druckzonen in Platten balken-, Dreick-, und Trapezeform)

R. Winders, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 9, Sept. 1960, pp. 210-214
Reviewed by Rudolph Szilard

Author derives formulas for the determination of the safety factor for plate girder, triangular, and trapeze forms. Monographs and three numerical examples facilitate the solution.

Bending of the walls of partly-full cylindrical tanks

J. D. Davies, *Concrete and Constructional Engineering* (London), V. 55, No. 10, Oct. 1960, pp. 377-379

Presents a simple means of calculating the stresses in the most adverse case of a partly-full tank, a condition not uncommon in some industrial plants. The partly-filled condition induces tensile bending stresses in the wall at the face in contact with the liquid. The wall thickness must be such that these stresses are not likely to cause cracking of the concrete. A tank designed for a full depth of liquid is generally sufficiently strong

to resist the effects of partial filling but if a tank cannot be filled above a specified level, economy may result if it is designed for the surface of the contents at this level.

Resistance of concrete piles to penetration

DONOVAN H. LEE, *Concrete and Constructional Engineering* (London), V. 55, No. 11, Nov. 1960, pp. 399-403

Reviews the various formulas for calculating the safe load on the pile and states their limitations as applied to concrete piles. A brief discussion is then had on the stress-wave theory. Finally a simple graphical method is presented based on this theory.

Wind stresses in domes

P. GONDIKAS and M. G. SALVADORI, *Proceedings*, ASCE, V. 86, No. EM5, Oct. 1960, pp. 13-29

AUTHORS' SUMMARY

The bending stresses due to antisymmetrical wind pressures in a hemispherical dome elastically built-in into a cylinder are determined by a Geckeler approach extended to nonsymmetrical loads. The simple form of the results allows their use by the designer. A numerical comparison with the asymptotic solution of Schwerin indicates the satisfactory accuracy of this approximate solution for all cases of practical interest. The methods used in the solution of the present problem are easily extended to spherical sector shells stiffened by rings at the boundary and to other types of nonsymmetrical loads.

Model methods, with particular reference to three recent applications in the fields of steel, composite and concrete construction

S. R. SPARKES and J. C. CHAPMAN, *The Structural Engineer*, V. 39, No. 3, Mar. 1961, pp. 85-99

AUTHOR'S SUMMARY

Available model methods are briefly described and examples are given from published work to indicate some of the design problems which have been solved with the help of models. A more detailed description is then given of the use of models at Imperial College by the authors to study respectively erection problems in a prestressed steel bridge, the stiffness and stresses in the reinforced concrete services tower of a tall steel framed building, and the stresses and deformations in a concrete arch dam.

Ultimate load design of statically indeterminate structures

English translation of *Bulletin D'Information* No. 21, Comité Européen du Béton, Paris, Jan. 1960, 39 pp. (Limited supply of single copies available from ACI headquarters.)

Another in the series of English translations of reports of the Comité Européen du Béton made available by the ACI-CEB Collaboration Committee.

This translation includes two papers: "Note on the Ultimate Load Design of Statically Indeterminate Structures," by Y. Guyon, and "Proposed Method of Analysis based on the Theory of Imposed Rotations," by G. Macchi. Appendix 1 contains comments on the proposed method of analysis, Appendix 2 proposes programs for immediate and long range investigations by the committee.

Structural engineering: Review and problems

NATHAN S. GLASSMAN and HARRY H. GRAEF, III International Publications, Washington, D.C., 2nd Edition, 1960, 96 pp., $7.50

Covers the analysis and design of many common structural members on various types of material — steel, concrete, and wood. Book is compact and condensed, with problems and text in close proximity. It should satisfy three major purposes — preparation for professional registration for architects; offers solutions for basic structural design problems in step-by-step sequence; and is a quick reference to design code requirements or for quick estimates of structural requirements. The book is written to be used in conjunction with the ACI Building Code, *Reinforced Concrete Design Handbook*, and the AISC handbook.

Total tensile force on openings in a uniaxial compression field (Die totale Zugkraft an Oeffungen in einem einachsigen (Druckspannungsfeld)

B. Cvetkov, *Der Bauingenieur* (Berlin), V. 34, No. 12, Dec. 1959, pp. 476-477
Reviewed by Aron L. Mirsky

Direct evaluation of an integral arising in the calculation of the total tensile force in uniaxial compression-fields with openings, as set up by Hiltscher and Pant [(*Der Bauingenieur*, V. 32, No. 12, Dec. 1957, pp. 470-474; "Current Reviews," ACI Journal, V. 30, No. 6, Dec. 1958 (*Proceedings* V. 54), p. 742] is demonstrated.

Determination of stresses from trajectory grids (in Dutch)

H. W. F. C. Heman, *Cement* (Amsterdam), V. 12, No. 11, Oct. 1960, pp. 979-987
Reviewed by John W. T. Van Erp

Graphic method of determining stresses from a given principal stress trajectory grid is described. The trajectory grid can be determined theoretically, but is mostly derived by experiment: photoelastic, brittle lacquers, strain gages. Derivation of stresses is shown starting from the equilibrium of a trajectory grid element. Examples of application are given. Accuracy of results is adequate, being 96 percent or better.

Foundations for rotating machines (in Dutch)

C. W. J. Groothoff, *Cement* (Amsterdam), V. 12, No. 9, June 1960, pp. 761-763
Reviewed by John W. T. Van Erp

The controversial subject of foundations for vibrating machines is treated with special emphasis on ever increasing number of revolutions and higher clearance requirements. Author states his preference for the flexible foundation, instead of the heavy rigid type. The axiom that the ratio of weight of superstructure plus machine over substructure should be less than one is apparently losing its validity. Examples are given of eight foundations for compressors, turbogenerators, etc., built during the recent years. The ratio for these is often as high as 2.8, which means a heavy superstructure on a light substructure. Example is given of a foundation structure with a three point support. This is located in a mining area with chances of heavy soil settlements, and the possible need for jacking up and realignment.

The "unified" influence diagram method—A new approach to analysis of plates

S. K. Mukhopadhyay, *Journal of Technology* (Calcutta), V. 4, No. 2, Dec. 1959, pp. 157-169
Author's Summary

Plates subjected to a haphazard pattern of loading, especially by a number of point loads, are conveniently analyzed by using the "specific" influence diagram for the case, which is a graphical representation of the variation of bending moment or twisting moment or any other required stress quantity, at a specified point of the plate of given proportions and boundary conditions. The analysis is rendered quite easy by this method, no doubt, but the presupposition is that the "specific" influence diagram is available, a presupposition which can hardly be fulfilled, except for a limited number of standard cases. Owing to this handicap, the influence diagram approach, therefore, cannot be used in most cases.

Object of this paper is to remove this handicap by presenting an alternative influence diagram approach, called here the "unified" influence diagram approach, as opposed to the conventional "specific" influence diagram approach. The method enables one to analyze rectangular plates of all possible side ratios and some triangular plates by using a single "master influence" diagram.

The method, it is hoped, will be useful to a practical designer by enabling him to analyze those plates, for which no "specific" influence diagrams are available, almost as easily as the standard cases.

Design and construction of reinforced concrete masonry columns and pilasters

National Concrete Masonry Association, Washington, D.C., 1960, 150 pp., $2.50

This book is designed to give structural engineers, architects, and builders, technical information on the design and construction of reinforced concrete masonry columns and pilasters. Gives the general characteristics and structural properties of reinforced concrete masonry and the general considerations for the design of columns and pilasters. Charts and tables are set up for allowable loads for different combinations of bars and eccentricities. Recommended construction practices and details are also given. The appendix summarizes reports of tests on concrete masonry columns and gives formulas and design factors from various building codes.

Recent trends in ultimate strength design

PHIL M. FERGUSON, *Proceedings*, ASCE, V. 87, No. ST1, Jan. 1961, pp. 57-71
AUTHOR'S SUMMARY

For reinforced concrete members, load-stress curves show such irregularities at cracking load and near ultimate load that the attainment of reasonably accurate predictions of member capacity requires the use of ultimate strength theory.

Under ultimate strength theory, the designer must become familiar with load factors (and probability theory). Several load factor proposals have been presented, each calling for the use of reduced material strengths and an increase over working loads. In addition, American study has always used separate load factors for dead and live load.

The increasing use of strains and the convenient use of the rectangular stress block for concrete stresses appear to furnish the tools that the designer needs to develop a systematic "theory" of ultimate strength for either complicated or standard sections.

Elastic analysis of a symmetrical fixed-end arch of varying section

A. F. GEE, *Civil Engineering* (London), V. 55, No. 653, Dec. 1960, pp. 1575-1581
AUTHOR'S SUMMARY

A fixed-ended arch is statically indeterminate to the third degree, and if, in addition, the arch rib is of varying section, it does not lend itself to direct analytical solution. Consequently, the derivation of influence lines for this type of structure is a laborious mathematical operation, and the amount of work involved is in almost direct proportion to the accuracy required.

The author therefore considered that the problem was ideally suited to the use of an electronic computer, and prepared the program describing it.

The rigid elliptic plate on elastic-isotropic foundation, with application to rectangular plates (Die starre elliptische Platte auf elastisch-isotroper Unterlage mit Anwendung auf rechteckige Platten)

KARL FISCHER, *Der Bauingenieur* (Berlin), V. 33, No. 3, Mar. 1958, pp. 97-101

Tilting of an elliptical plate on homogeneous foundation through an eccentric load, with application to rectangular plates (Die Schiefstellung einer elliptischen Platte auf homogener Unterlage durch eine ausmittige Last mit Anwendung auf Rechteckplatten)

KARL FISCHER, *Der Bauingenieur* (Berlin), V. 34, No. 7, July 1959, pp. 258-265
Reviewed by ARON L. MIRSKY

While these papers are primarily in the field of soil mechanics, the contents will be of considerable value to structural engineers. First paper treats cases of axial (concentric) load; results are extended in second paper to case of eccentric loading, and tilt (inclination) of plate is investigated. Results are applied to rectangular plates by converting them to equivalent elliptical plates (height-width ratio = ratio of minor-major diameters; and equal areas). Numerical examples are given.

Calculation of thin shells of revolution of variable thickness

J. Rygol, *Civil Engineering and Public Works Review* (London), V. 55, No. 649, Aug. 1960, pp. 1015-1020

Author's Summary

The calculation of symmetrically loaded shells is generally achieved by superimposing on the membrane forces a system of horizontal forces and moments uniformly distributed along the parallel edge circle.

Paper describes a practical method of calculating shells of variable thicknesses and an "exact" solution is offered for a shell where the square root of the shell thickness decreases linearly with the distance from the edge.

Data book for civil engineers. Volume 1 — Design

Elwyn E. Seelye, John Wiley & Sons, Inc., New York, 3rd Edition, 1960

This edition of this popular reference book has been updated to include dedevelopments and improvements in various materials and methods of construction and design applications. In the concrete sections tables for axial and eccentric loads on both uncracked and cracked sections for columns have been expanded to cover a wide range of sizes and concrete strengths all in accordance with the ACI Building Code. New sections cover prestressed concrete and ultimate strength design. Composite beam design methods are given. Some information given on physical properties of lightweight concrete. All design, examples, and charts, are based on the latest ACI Building Code.

Advances in design methods with other materials are also given. Likewise information relative to design and construction of special applications such as airports, bridges, and waterfront structures are included.

This is a concentrated collection of data helpful to architects and engineers to design, place under contract, and construct all types of civil engineering structures.

Design of shell structures—Folded plate forms

Milo S. Ketchum, *Consulting Engineer*, V. 16, No. 1, Jan. 1961, pp. 118-122

Reviewed by Aron L. Mirsky

Brief descriptive catalog of the basic forms and shapes and what can be done with them, well illustrated with sketches and some studied photographs.

Design of biaxially-loaded columns by the ultimate-load method

F. N. Pannell, *Concrete and Constructional Engineering* (London), V. 55: No. 10, Oct. 1960, pp. 365-370; No. 11, Nov. 1960, pp. 409-417; No. 12, Dec. 1960, pp. 443-446

Presents design charts for columns with nearly equal amounts of reinforcing on each side. Part 1 considers columns with 12 or more bars; Part 2 covers columns with eight and four bars. Each of the cases is illustrated by examples; Part 3 forms the appendix to the series presenting the equations of static equilibrium for the various cases.

Design of structures

English translation of *Bulletin D'Information* No. 10, Comité Européen Du Béton (Paris), Jan. 1959, 33 pp. (Limited supply of single copies available from ACI headquarters.)

This bulletin presents three of the documents which were developed at a scientific conference held in Moscow under the auspices of the International Building Council to which the Academy of Architecture of the USSR invited members of the CEB. The first paper compares the thinking of Russian specialists with that of the European Committee and American research workers. The second is a reply by the president of CEB which states the position of the CEB and voices a wish for the establishment of closer cooperation between the western countries and the USSR in matters bearing on civil engineering problems. The third paper is a resolution which includes a listing of research activities showing the Soviet centers and those of other countries interested in the various problems.

Materials

What happens to aluminum in heavy-aggregate concrete?

R. S. Dalrymple, *Modern Concrete*, V. 24, No. 5, Sept. 1960, pp. 56-58

Contains the results of a test through 12 months exposure for aluminum which is cast in magnetite aggregate concrete.

Aluminum alloys cast in magnetite aggregate concrete and maintained dry are subject to only slight attack during the setting period. This attack does not continue with increasing exposure.

Aluminum alloys cast in magnetite aggregate concrete and maintained wet are subject to more aggravated attack, which increases in severity with time during the first 3 months. Thereafter the attack appears to cease.

Some alloys are more resistant to exposure in magnetite aggregate concrete than are others.

Investigation of nonmetallic waterstops. Preliminary laboratory and field exposure tests

B. J. Houston, *Technical Report* No. 6-546, Report 1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., May 1960, 25 pp.

Authors Summary

Four types of rubber, polyethylene, and polyvinylchloride (PVC) were subjected to laboratory tests and exposed to natural weathering to obtain comparative data on their performance as waterstops. Results showed that: The PVC's and polyethylene tested were generally not as strong in tension nor as elastic as the natural, service, and neoprene rubbers tested, but were stronger and more elastic than the butyl rubbers tested, They were harder than the rubbers. PVC and butyl absorbed almost no water compared to 6 to 16 percent absorbed by natural, service, and neoprene, and were not materially affected by oxygen under pressure and heat whereas the other rubbers were. PVC did not lose plasticizers in presence of alkalies. It had almost as much elastic recovery at low temperatures as rubber, but recovered much slower. It was not adversely affected by weathering whereas natural and service rubbers were, and neoprene exposed under stress relaxed. Corps of Engineers specifications for PVC waterstops are included as Appendix A. Additional tests are proposed.

Water and air in mortars and concrete (in Dutch)

Fritz Keil, *Cement* (Amsterdam), V. 12, No. 10, Aug. 1960, pp. 847-851

Reviewed by John W. T. Van Erp

Modern concrete technology considers the four concrete components: aggregate, cement, water, and air as equally important. This paper investigates the last two more thoroughly. Surface tension and its effects are investigated, also its reduction by plastifiers. It is demonstrated that the film-forming quantity of water, called "pseudo-solid water," will coat all solid particles with a coat of only a few molecules thickness and there should be no excess water in the capillary pores, which would create opportunities of chemical and physical attack of the concrete. Optimum water cement ratio is derived as 0.4, excess volume filled by air only.

Pavements

Paving Methods for the Salzburg-Vienna Autobahn

Helmut Mayer, *World Construction*, Apr. 1960, pp. 29-32

Reviewed by Paul J. Fluss

The Autoban is built as a divided highway. Each roadway has a width of 24.60 ft (7.50 mr) plus border strips, shoulders, and green strips. The pavement consists of air-entrained concrete, having a cement content of 5.56 sacks per cu yd(330 kg per cu m). It is built in two layers of 6.3 (16 cm) and 2.36 in. (6 cm) and reinforced with 0.217 in. (5.5 mm) diameter wire mesh. Small granite stone blocks on a concrete base are used for the border strips and bituminous construction methods

for the shoulders as well as for the approaches and junctions. Starting with the foundation and design problems, the well illustrated article gives a detailed description of the materials for proportioning, production, control, placing, and finishing of the concrete as well as of the achieved results. Completion of the Autobahn is scheduled for 1961.

WRI airport concrete pavement handbook

Technical Committee, Wire Reinforcement Institute, Washington, D. C., 1960, 96 pp., $3

Covers these major topics — basic factors in design; concrete pavement design fundamentals; subgrade, subbase and drainage; pavement thickness; reinforcement design; construction practices; concrete overlays; advantages of reinforcement; and welded wire fabric data.

Also included are tables of aircraft loadings and landing gear details of current aircraft and specifications for wire fabric. Sketches show typical layouts of runways, intersections, reinforcement, and joint details. Job pictures show proper construction practice.

Anti-slip properties of concrete roads (in Dutch)

C. VAN DE FLIERT, *Cement* (Amsterdam), V. 11, No. 5, Oct. 1959, pp. 418-421
Reviewed by JOHN W. T. VAN ERP

All thruways in the Netherlands are inspected for anti-slip properties every summer. Special measuring vehicle has been developed to take rapid readings. Conclusions drawn from these inspections over a number of years are: (1) gravel is better aggregate than limestone for permanent ruggedness; (2) brooming of fresh concrete surface is adopted as standard practice to remove smooth cement paste and expose aggregate; (3) curing always 3 weeks at least; membrane curing by curing compounds not proven satisfactory; (4) concrete roads somewhat better than bituminous and much better than brick pavement; (5) ruggedness of worn and polished concrete roads restorable by muriatic acid treatment.

Precast Concrete

Prefabricated prestressed crane girders for E.N.C.I. cement plant (in Dutch)

J. J. ENDTZ and C. L. JONGE, *Cement* (Amsterdam), V. 12, No. 11, Oct. 1960, pp. 944-946
Reviewed by JOHN W. T. VAN ERP

Comparative design in structural steel proved prestressed concrete most economic. Favorable experience with similar previous crane girders also determined selection of design. Girders were cast upside down on the ground, taken out of forms after 2 days and placed on curing bed. After prestressing girders were raised 70 ft and placed on columns 30 ft on center. Track rails rest on rubber-covered steel plates directly on the concrete. Finished job provided quiet crane operation due to rigidity of girders.

Test of joints subjected to bending and shearing in precast members

W. J. LARNACH, *Concrete and Constructional Engineering* (London), V. 55, No. 12, Dec. 1960, pp. 433-439

Describes the behavior of beams with several types of joints comparing them with equivalent beams without joints to assess the efficiency of the joints in transmitting bending moments. The types of joints used were square scarf, sloping scarf, doweled, covered butt, and a second type of sloping scarf with three bolts.

It was concluded that the introduction of a joint materially reduces the stiffness of a beam. The doweled and covered butt joints affected the stiffness least. In bolted joints a significant change in load appears to occur only on those bolts situated in the "top half" of the joint. If large bending moments are to be transmitted with this type of joint it is advisable to use high tensile bolts which would permit the use of large initial clamping forces while

avoiding the possibility of premature failure in the joint due to early yielding of the bolts. Also the effective area of concrete would be increased due to the use of smaller bolts.

The presence of a considerable shearing force did not appear to affect the efficiency of the sloping scarf joint. It is suggested that additional reinforcement against shearing be provided in the region of joints under the action of shearing force. Extremely high efficiencies can be achieved with doweled and butt joints. These joints would probably be more difficult than the others to make on site but may be more economical if the transmission of higher bending moments is necessary.

Prestressed Concrete

Prestressed concrete simply explained

H. KAYLOR, Contractors Record, Ltd., London, 1961, 158 pp., 28s.

The theme of the book is well set forth in its title. The first half of the book explains in everyday language the basic principles of prestressed concrete, describes methods and applications (mostly British), materials, and allowances for losses of prestress. A chapter is devoted to design of simply supported beams followed by examples of design of pretensioned and post-tensioned units. Composite action between prestressed elements and cast-in-place slabs in bridges and buildings is analyzed.

Prefabricated factory building with prestressed lightweight roof elements (in Dutch)

Cement (Amsterdam), V. 12, No. 10, Aug. 1960, pp. 881-884

Reviewed by JOHN W. T. VAN ERP

For a concrete products factory a building of 100 x 330 ft was built entirely of prefabricated elements of their own manufacture. Column spacing is 33 x 50 in., the 33 ft span being bridged by hollow prestressed girders, separated by strips for lighting. These girders were composed of 7 ft long units made of Hollith concrete, which is a lightweight concrete using similar aggregates to sintered carbonaceous shale. The girders were cast on the ground in steel forms for their full length, with thin metal separators, thus subdividing them into the 7 ft lengths, and also making a perfect fit without any grout on assembly. Wall units were slabs, 6 in. thick, of which the inner 4 in. were also of lightweight concrete.

Fire-resistance of prestressed concrete beams

LESLIE A. ASHTON and STEPHEN C. C. BATE, *Proceedings*, Institution of Civil Engineers (London), V. 17 (Session 1959-60), Sept. 1960, pp. 15-38

Reviewed by ARON L. MIRSKY

Tests were conducted on simply supported, prestressed concrete beams with post-tensioned cables and composite slab, to obtain information for design of beams for buildings of high fire risk (e.g., large warehouses). Results indicate that for such structural members of normal type, resistance to fire is governed by the protection afforded the prestressing steel; 6-hr ratings can be attained. Failure of beams carrying their design loads occurs when mean temperature of the prestressing steel reaches about 400 C; for temperatures above about 180 C, the reduction in stiffness will usually necessitate replacement of the member.

Design of prestressed concrete beams

WILLIAM H. CONNOLLY, F. W. Dodge Corp., New York, 1960, 252 pp., $11.50

A rational and clear-cut method covering the most difficult and time-consuming aspect of prestressed concrete design: selecting the cross section. Author approaches the problem of selecting the cross section in a logical, straightforward manner. He places major emphasis on elastic design for bending, gives consideration to prestress losses, shear, and ultimate flexural strength; and uses the report of

ACI-ASCE Committee 323 "Tentative Recommendations for Prestressed Concrete" as a guide in considering these important factors.

The 120 tables are adaptable to members of any proportions and dimensions. Through their use, a designer may reduce trial and error normally involved in designing these members. A useful handbook for structural engineers, architects, civil engineers, and professors of engineering and architecture. Academic frills are completely eliminated; the problem is treated from a strictly practical point of view.

Properties of Concrete

Predictable strength of young concrete

R. NARAYANAN, *Engineering* (London), V. 189, No. 4906, Apr. 29, 1960, pp. 580-581; Discussion, May 27, 1960, p. 713

Reviewed by ARON L. MIRSKY

Reports results of tests which indicate that at all temperatures below 158 F, the maturity-strength *pattern* is independent of the curing temperature, Plowman's Law being valid for curing temperatures below that value, and that the earliest age at which a reasonably accurate prediction of long-term strength is possible corresponds to a maturity of 2400 deg-hr.

Validity of results has been questioned by T. N. W. Akroyd (*Engineering*, May 27, 1960, p. 713), primarily on basis of author's use of oven rather than water bath for curing.

Using sands conforming to BS 882: 1954 in a study of the voids in mortar

W. J. LARNACH, *Civil Engineering and Public Works Review* (London), V. 56, No. 655, Feb. 1961, pp. 194-196

AUTHOR'S SUMMARY

By carrying out a standard compaction test on mortars having a wide range of richness, water content, and sand grading, it is found that each grading exhibits different characteristics. By calculating the voids-cement ratios for mortars at optimum water content (corresponding to minimum voids content) and relating these values to compressive strength, a tentative method for comparing the mortar-making (and possibly the concrete-making) properties of the sands is suggested.

Effect of retarders on setting time and strength

R. D. GAYNOR, *Cement, Lime, Gravel*, V. 34, No. 7, 1959, pp. 196-200

ROAD ABSTRACTS
V. 27, No. 8, Aug. 1960

The work reported was carried out at the University of Maryland to study the effects of using four different retarding admixtures on (1) the water requirement and strength of concrete and (2) the rates of setting and hardening of mortar. Results are tabulated and shown graphically. The retarders improved concrete strength and may therefore be of value for ready-mixed concrete in hot weather.

Can concrete strength be effectively measured by ultrasonic methods? (in Russian)

S. S. GORDON, I. V. ZASHCHUK, and G. F. NADAREISHVILI, *Avtom. Dorogi* (Moscow), V. 23, No. 2, 1960, pp. 5-13

HIGHWAY RESEARCH ABSTRACTS
V. 31, No. 3, Mar. 1961

An account is given of research carried out at the concrete and radioelectronic laboratories of the Reinforced Concrete Research Institute of the USSR. Tests with the ultrasonic impulse meter PIK-5a indicated that at present the ultrasonic method of measuring the strength of concrete of unknown specification cannot be considered to be reliable. The method is accurate to ± 10 percent when comparative data on given concrete are available. Instruments of the PIK-5 type can be satisfactorily used to check the strength of prefabricated concrete products of a given type and of concrete road pavements and to measure the increase in concrete strength with aging.

Determination of tensile strength of concrete and mortar by the split test

C. K. RANESH and S. K. CHOPRA, *The Indian Concrete Journal* (Bombay), V. 34, No. 9, Sept. 1960, pp. 354-357

Reports results of experiments by the "split test" on mortars and concrete for the determination of tensile strength and discusses the theory underlying the test. Results were compared with those of other investigators. In view of the simplicity of the test and the lower values of coefficient variation than other common tensile tests, authors recommended adoption of the test in the Indian Standards.

Detection of internal and surface cracks in concrete with brittle electric conductors made of graphite (in German)

C. ROHRBACH and E. JUNG, *Materialprüf*, V. 1, No. 11/12, 1959, pp. 395-399

ROAD ABSTRACTS V. 27, No. 6, June 1960

The apparatus for internal crack detection essentially consists of brittle graphite rodlets which fracture simultaneously with the concrete, causing a variation of conductivity which can be measured electrically at any distance with simple instruments. Surface crack detection is achieved by covering the concrete with a thin, brittle graphite layer whose conductivity is altered when cracking occurs. Both detectors permit accurate measurements of cracks with widths of 0.1 mm and above.

Automatic equipment for rapid freezing and thawing of concrete in water

HERBERT K. COOK, *Bulletin* 259, Highway Research Board, pp. 7-27

AUTHOR'S SUMMARY

Describes construction and operation of an automatic, 25-specimen capacity, freezing-and-thawing device for the testing of standard concrete specimens. The cycle employed is rapid freezing and thawing in water in accordance with AASHO T161—ASTM C290, which is the method specified for the evaluation of both air-entraining admixtures for concrete and air-entraining additions for the manufacture of air-entraining portland cement.

Details of construction, instructions for operation and maintenance, and a list of component parts with their individual costs and over-all construction cost, are provided in appendices.

Experiences with operation of the equipment over 4 years and typical test results obtained on concrete specimens are discussed.

Crack formation due to shrinkage of reinforced concrete

SVEN T. A. ODMAN, *RILEM Bulletin* (Paris), No. 9, New Series, Dec. 1960, pp. 100-109

AUTHOR'S SUMMARY

The process of deformation in the course of drying of a reinforced concrete prism is described in the introduction which is followed by a theoretical study of this problem.

The difference between the strains of nonreinforced and reinforced concrete is regarded as a fictitious tensile strain of the concrete, and is assumed to be proportional to the ratio of reinforcement and to the compressive strain of the reinforcement. Owing to the introduction of the concept of "effective bond force" in any arbitrary cross section of the prism, the relative slip between steel and concrete can also be expressed by a function of the compressive strain of the reinforcement. Finally, the actual tensile stress in the concrete on a level with the center of gravity of the reinforcement is supposed to be proportional to the corresponding tensile stress in the concrete deduced from the theory of elasticity.

These assumptions have served as a basis in the deduction of formulas for steel stress, concrete stress, crack spacing, and crack width, all of them being functions of the shrinkage of the same nonreinforced concrete prism. All unknown coefficients entering into these formulas were numerically evaluated as functions of the drying time

by an analysis of some test results published by R. Dutron in 1934. The paper closes with a discussion of its results.

Comparison of various methods of capping brick, structural tile, and concrete block for compressive strength tests

C. M. Dodd and T. D. McGee, *Engineering Report* No. 32, Iowa Engineering Experiment Station, 1960, 22 pp., $0.50

Author's Summary

Comparative tests of various capping materials were conducted to determine the effect of the capping on the apparent compressive strength of building brick, structural clay tile, and concrete block. After statistical analysis the results indicate that it is possible to use compressible materials such as insulating wallboard as the cap for building brick and vertical-cell concrete block. Extensive tests throughout the appropriate industries should be made to obtain sufficient data for an industry wide statistical analysis and general conclusions on the suitability of deformable capping materials for compressive strength tests.

Investigation concerning the compaction of concrete on vibrating tables with fixed and loosely set-on forms (Untersuchungen über das Verdichten des Betons auf Rütteltischen in aufgespannten und lose aufgesetzten Formen)

K. Walz, *Beton-Herstellung und Verwendung* (Düsseldorf), V. 10, No. 6, June 1960, pp. 270-272

Reviewed by Ferdinand S. Rostasy

This laboratory investigation deals with the efficiency of compaction of vibrating tables measured in terms of the compressive strength. Under otherwise identical conditions the forms were either fixed to or loosely set on the table. Two concrete mixes were investigated and different kinds of vibrations (vertical or circular motion of the table) were included. It was found that for either concrete or type of vibration the loosely set-on forms were better compacted. Considerably greater accelerations were necessary for the same compaction time to obtain the same density and strength. The practical implications are that loosely set-on forms are more efectively compacted on vibrating tables constructed only to exert small accelerations.

Plastic deformations of beams due to impulsive loading (in Japanese)

Hiroshi Yonezawa, *Transactions*, Japan Society of Civil Engineers (Tokyo), V. 22, No. 61, Mar. 1960, pp. 10-17

Reviewed by Kiyoshi Okada

Describes the theoretical analysis of the plastic deformation of beams having one fixed and one simply supported end. Discusses two kinds of loading, one uniformly distributed and one concentrated, and obtains a formula for the permanent deformation of beams.

Time-dependent deformations of plain concrete

John R. Keeton, *Proceedings*, Highway Research Board, V. 39, 1960, pp. 310-335

Author's Summary

Cylindrical concrete specimens in sizes 3 x 9 in., 4 x 12 in., and 6 x 18 in. were subjected to constant sustained compressive stresses of 2000 and 3000 psi, representing 38 and 57 percent, respectively, of the ultimate compressive strength at time of loading. Specimens were placed in controlled environments of 20, 50, and 100 percent relative humidity.

Concrete deformation measurements were made in the central portions of the specimens with mechanical strain gages and end-to-end with electronic cantilever deflection reeds. Shrinkage measurements were made on unloaded companion specimens.

Test results are reported for 175 days of loading. Creep strain did not appear to be greatly influenced by size in the range of sizes used. Creep was 5 to 10 percent greater in 20 percent than in 50 percent relative humidity.

Effect of alkali on the strength of portland cement paste

K. M. Alexander and C. E. S. Davis, *Australian Journal of Applied Science*, (Melbourne), V. 11, No. 1, 1960, pp. 146-156

Compressive strengths and moduli of rupture were determined in portland cement pastes containing up to 2.8 percent of added alkali. A range of different water-cement ratios, ages, cements, and types of alkali was studied.

At all ages and water-cement ratios, the addition of a given amount of alkali caused almost the same percent reduction in compressive strength over the whole range of water-cement ratios. The factors of water-cement ratio, alkali content, and age did not interact significantly.

The modulus of rupture was decreased by small additions of alkali, increased by further additions, and finally decreased again. The main influence was the quantity of added alkali but the other factors did interact to some extent.

Alkali had essentially the same effect on the strengths of different cements. No difference was detected between added hydroxides of sodium and potassium.

Variation with time of tensile and compressive strength of hydraulic concrete (in Russian)

I. G. Ginsburg and R. E. Litvinova, *Gidrotekhnicheskoe Stroitel* (Moscow), V. 28, No. 2, Feb. 1959, pp. 30-33
Applied Mechanics Reviews V. 13, No. 12, Dec. 1960

Authors present results of tests conducted at the BNIIG concrete laboratory. Tensile specimens had a critical cross section of 5 x 5 cm; compressive specimens were 7 x 7 x 21 cm or 10 x 10 x 10 cm. Portland cement was used. Water-cement ratio ranged from 0.5 to 0.7. Age of test was 3, 7, 28, 180, and 360 days.

Authors investigate the relation existing between tensile and compressive strength of mortar and of concrete for various W/C ratios and ages. Strength increases with time and decreases as W/C becomes larger. For concrete the ratio of tensile strength to compressive strength is not a constant. For ages from 3 to 28 days it decreases markedly with time. In general the higher W/C ratios correspond with the higher values of the tensile-compressive strength ratio.

The ratio of tensile strength of mortar to that of concrete varies also with time. At early ages ratio is > 1.0 (up to 1.6). As age increases, ratio decreases and at a year is < 1.0 (about 0.9).

Finally authors call for standardization of specimens and tests and for consideration of test results in design.

Contribution for knowing the influence of plasticizing agent on the creep and shrinkage of concrete

F. Peres Rodrigues, *RILEM Bulletin* (Paris), No. 6, New Series, Mar. 1960, pp. 39-46
Author's Summary

The influence of the addition of a plasticizing agent on the creep and shrinkage of a concrete is studied. Additionally, the influence of the plasticizer on the coefficient of hygrometric permeability and on the compressive rupture strength in function of the relative humidity of the storage atmosphere was also determined.

Studies showed that the addition of the plasticizing agent decreased by about 20 percent in four prisms loaded at 28 days. The shrinkage tests were carried out in atmospheres with relative humidities of 35, 55, 75, and 100 percent (saturated chamber). The influence of the plasticizer is not marked in the case of shrinkage but is considerable on the coefficient of hygrometric permeability which is increased all the more the lower the relative humidity. Hence the fact that the decrease in rupture strength induced by the relative humidity of the storage atmosphere is more marked for concrete with admixture although the rupture strength of this concrete always exceeds that of concrete without plasticizing.

Skeleton strength and critical porosity in set sulphate plasters

K. K. SCHILLER, *British Journal of Applied Physics* (London), V. 11, No. 8, Aug. 1960, pp. 338-342

Reviewed by ARON L. MIRSKY

Strength of sanded plasters is shown to be equal to that of the matrix cementing the sand grains together, the sand grains being regarded as solid pores. Strength decreases as amount of sand increases, due to the addition of more water to maintain the desired consistency (the "water demand"). Griffith's theory of the strength of a brittle material as affected by the presence of small cracks is shown to apply to the "quality factor," a parameter affording a measure of the skeleton strength. Critical porosity is found to be the *total* porosity (voids plus sand) at which the strength becomes negligible.

Strength of concrete in specimens of various shapes and dimensions (La resistance du beton etudiee sur des eprouvettes de differentes formes et de diverses dimensions)

W. KUCZYNSKI, *RILEM Bulletin* (Paris), No. 8, New Series, Sept. 1960, pp. 77-92

Report of a study by the Polish Committee on Standards of the influence of form and dimensions of concrete specimens on strength. Results of the tests, without comment or modification, are given in tabular form.

Movements in concrete during the period following its compaction in a mold

J. D. MCINTOSH, *RILEM Bulletin* (Paris), No. 9, New Series, Dec. 1960, pp. 6-10

AUTHOR'S SUMMARY

The implications of the vertical and horizontal movements which take place in concrete during the period immediately after casting are discussed in the light of results which have been obtained using the "early shrinkage apparatus." It seems that these movements may account for appreciable variations in the pore structure of hardened concrete near horizontal surfaces and that standard laboratory procedures for making test specimens might well be reviewed if more reproducible results are to be obtained in tests in which the migration of water through concrete plays an important part.

Compressive strength of concrete—cube or cylinder? (in Norwegian)

HENRY HANSEN, ARVID KIELLAND, KNUD E. C. NIELSEN, and SVEN THAULOW, *Nordisk Betong* (Stockholm), No. 4, 1960, pp. 305-320

Reviewed by MARGARET CORBIN

A review of the pros and cons of the two types of test specimens, cubes and cylinders, concludes that friction between the loading surfaces is the primary cause of the varying relationship between cube and cylinder strength. This relation is also affected by deformations in the heads of the testing machine. However, both sources of error can be eliminated. Friction can be counteracted by making the specimen sufficiently tall ($h/d = 2.0$); test head errors can be avoided by making the heads sufficiently rigid. Advantages of cylinder specimens are enumerated.

Study of creep of dam concrete and its application to the measurement of the behavior of dams (in Japanese)

HIROTSUGU, KIMISHIMA, *Transactions*, Japan Society of Civil Engineers (Tokyo), No. 72, Jan. 1961, pp. 23-30

Reviewed by KIYOSHI OKADA

Describes the creep tests of concrete made by using various sizes of specimens all sealed as mass-cured, and analyzes stress distributions by using the deformations measured in some dams by applying the test results. Concludes that (1) creep of mass concrete can be estimated by that of small specimen made of wet-screened concrete when taking into account the unit cement contents to which the creep values are proportional, and (2) creep value is expressed by $\varepsilon_{total} = 1/E + F(k) \cdot \ln(t + 1)$ where (E, $F(k)$ = constant, t = load duration in days).

Experiments on concrete bars. Expansions during storage in climate room

AXEL ERSEN and OLE GLARBO, *Progress Report* K1, Committee on Alkali Reactions in Concrete, Danish National Institute of Building Research and the Academy of Technical Sciences, Copenhagen, 1960, 38 pp., 12 D Kr.

Reports the results of concrete prism tests containing varying proportions of alkali reactive aggregates.

Durability of reinforced concrete in sea water

Research Paper No. 30, National Building Studies, Department of Scientific and Industrial Research, H. M. Stationery Office, London, 1960, 42 pp. 4s

Twentieth and final report of the Sea Action Committee of the Institution of Civil Engineers summarizes an investigation begun in 1929 on the resistance of reinforced concrete to attack by sea water. A total of 540 piles and 8000 cylinders were exposed to normal and concentrated sea water in two locations in England. The concrete mixes correspond to 1000, 600, and 360 lb of cement per cu pd of concrete which had slumps of ½ and 2 in. in various series. Cover over reinforcement was either 1 or 2 in.

Observations on cracking of the piles, corrosion of reinforcement in uncracked piles, and deterioration of the concrete cylinders gave a broadly consistent picture of the effect of cement content of the concrete, cover over reinforcement, and type of cement. With portland and trass-portland cements the general order of decreasing resistance to cracking was:

1. Rich (1:2⅔) mix piles with 2 in. cover
2. Medium (1:5) mix piles with 2 in. cover
3. Rich (1:2⅔) mix piles with 1 in. cover
4. Medium (1:5) mix piles with 1 in. cover
5. Lean (1:9) mix piles with 1 in. or 2 in. cover.

With high-alumina and portland blast furnace cements there was little difference between (2) and (3) with some tendency for a reversal of their order.

Influence of fine-aggregate grading on properties of concrete — A review of literature

W. O. TYNES and K. L. SAUCIER, *Technical Report* No. 6-544, *Report* 1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., May 1960, 11 pp., $0.50

AUTHOR'S SUMMARY

A literature search revealed that in most investigations dealing with the amount of material passing the No. 100 sieve that will produce satisfactory concrete, it was indicated that less than 15 percent of this size material is desirable, and that 10 percent is optimum for mass concrete. The consensus was that fines cause an increase in the water requirement and influence air content. One of the studies reported indicated that high air content did not decrease concrete strength excessively and seemed to be beneficial. Thus, it appears that a mass concrete mixture low in fines and having a high air content may prove satisfactory. It was generally concluded that the fine-aggregate grading influences in varying degrees the workability, durability, strength, water requirement, air content, bleeding, and density of concrete. It was also found that satisfactory concrete could be produced with a sand of which 5 percent passes the No. 50 sieve and none passes the No. 100 sieve. Thus, it appears that good quality concrete can be produced with fine aggregate deficient in certain sieve sizes.

An accelerated test for the 7- and 28-day compressive strength of concrete

J. W. H. KING, *Journal of Applied Chemistry* (London), V. 10, Part 6, June 1960, pp. 256-262

HIGHWAY RESEARCH ABSTRACTS V. 30, No. 8, Sept. 1960

As it is usually impossible to remove work made of concrete not up to standard at an age of 28 days (because there is much more recent work on top of it), in place of the present-day standard test for concrete, viz., the crushing strength of sample cubes 28 days after casting, a test has been evolved to predict the 7- and 28-day

strengths in as little as 7 hr. The sample cubes are subjected to a specified heating procedure, after which their immediate crushing strength can be used to predict the value at 7 or 28 days, by curves established experimentally. These have been checked with a wide range of concrete mixes, and with a wide range of cements, and in general, predict with a standard deviation of less than 200 psi at 7 days and 225 psi at 28 days. This is in general more consistent than the results at 28 days on site samples of what is reputedly the same concrete.

Effect of surge currents on the mechanical properties of concrete for overhead transmission line supports

A. V. KORSUNTSEV and S. D. MERKHALEV, *Elektricheskiye Stantsii* (USSR), V. 29, No. 8, 1958, pp. 73-76 (English translation No. 59-22454 available from Library of Congress or SLA Translation Center: microfilm $1.80, photocopy $1.80)

TECHNICAL TRANSLATIONS
V. 3, No. 10, May 25, 1960

Concrete foundations for supports can be used as earthing without additional metallic earthing but can be permitted only when the foundation is certain to be wet throughout the whole stormy season. If such a procedure is necessary, the mean current density at the surface of the reinforcing rods and other metal parts in contact with the concrete should be calculated from the calculated lightning current. This mean current density should not exceed 1 to 2 amp per sq cm. The parts of aboveground structural elements should not have any interruptions in the current-carrying path, filled with concrete.

Thermal treatment of lightweight concrete (Warmbehandlung von Leichtzuschlagbetonen)

SIEGFRIED REINSDORF, *Bauplanung-Bautechnik* (Berlin), V. 13, No. 10, Oct. 1959, pp. 471-475
Reviewed by J. F. LEPPMANN

Description of laboratory tests made by the Institut für Baustoffe, Weimar, to determine the influence of humidity, temperature, and timing of warm air curing on early and late strength of four types of lightweight concrete. It was found that the capacity for early high strength development increased with the porosity of the aggregate, but that there was little difference in the 28-day strength of the investigated types of concrete. Best results were obtained with air of 158 to 176 F and 80% relative humidity.

Creep and stress relaxation of concrete

TORBEN C. HANSEN, *Proceedings*, Swedish Cement and Concrete Research Institute at the Royal Institute of Technology, Stockholm, No. 31, 1960, 112 pp.

The first chapter deals with the definition of creep.

In the second chapter, three practical problems concerning the design of creep tests are dealt with: planning and evaluation of creep tests on a statistical basis, influence of load eccentricity on creep tests, and influence of shrinkage stresses on creep.

The concept of "basic creep" is introduced in Chapter 3. Basic creep is defined as creep when no moisture diffusion takes place in the concrete while loaded. It is asserted that any empirical connection between basic creep and the different factors that influence creep can be directly analyzed in a theoretical way using volumetrical or rheological considerations, since the results obtained from tests have not been distorted by the effects of drying or wetting on the various concretes. It is therefore more likely that a possible connection between creep and the factors influencing creep should be revealed if basic creep has been determined by creep tests, than if creep of drying concrete has been determined.

Following the same lines in Chapter 4 the two components of creep, delayed elasticity and viscosity, could be related to the gel density at any age of the cement paste, and to the volume concentration of cement paste in concrete. Even the effect of time under sustained loading could be taken into consideration when applying certain

standard time functions for the two components of creep (established on the basis of a rheological model).

The sum of the theoretical expressions for delayed elasticity and viscosity gave the final creep-equation for concrete, not exposed to any drying or wetting during the time of sustained loading (basic creep). It has been shown that the creep calculated according to this equation is in good agreement with the creep determined in 42 creep tests found in the literature.

In the same chapter the effect of revibration on creep has been investigated and was found to be insignificant; on the other hand, influence of temperature is considerable. The influence on deformational properties of normal aggregate materials is without any importance, whereas the effect of water permeability of aggregates is significant.

In Chapter 5 the influence of drying on creep of concrete has been dealt with.

Creep of cement mortar beams has been investigated while the beams were exposed by cyclic variations in the relative humidity of the surroundings.

Finally a tentative explanation of the effect of drying on creep is given in Chapter 5.

Results obtained at the C.B.I. (reported in Chapter 6) have shown that the nonlinear or "plastic" behavior of concrete at high overload is due to the formation of internal microcracks in the material. Plasticity is therefore a sign of initial rupture in the concrete. In Chapter 6 it has also been shown that plain mortar beams submitted to a bending moment get deep transversal cracks in the tensioned zone at a stress which is much below the ultimate strength. In an experimental investigation the plastic limit of concrete was determined at early ages. It was found that the plastic limit is low during the first 2 days after casting. A better concrete quality raises the level of the plastic limit at a higher rate, but the final level remains unchanged and independent of the quality.

In the last chapter a new method for calculating the stress relaxation on the basis of concrete creep data has been proposed. This method is in good agreement with experimental results.

Structural Research

Punching of concrete slabs without shear reinforcement

S. KINNUNEN and H. NYLANDER, *Transactions*, Royal Institute of Technology, Stockholm, No. 158 (Civil Engineering No. 3), 1960, 112 pp., 11 Kr.

Describes experimental and theoretical studies dealing with punching of concrete slabs in the vicinity of columns.

The experimental investigation was made on circular concrete slabs, 15 cm thick, which were subjected to loads along the circumference, and were supported on a column stub at the center. The slabs were tested to failure. The variables in these tests were the type of reinforcement and the amount of flexural reinforcement employed for each type of reinforcement. The tests comprised measurements of deflections, strains in the reinforcing bars, and concrete strains on the bottom surfaces of the slabs. In addition, the load and the appearance of the first shear was determined.

The test results are discussed in detail. In all the alternatives covered by the tests, the first shear crack opened up at a load which ranged from 45 to 75 percent of the ultimate load. In the main, only radial cracks were observed in the slab portion situated outside the shear crack. This corresponded to the fact that the deflection measurements showed a linear increase in the deflection of the slab portion outside the shear crack. In view of these observations, the theory expounded in this bulletin was based on the assumption that the slab portion outside the shear crack, which is bounded by this crack, by radial

cracks, and by the circumference of the slab, can be regarded as a rigid body, which is turned under load action around a center of rotation located at the root of the shear crack.

The theory advanced was found to be in satisfactory agreement with the test results. The theory explains the increase in the punching load with the amount of reinforcement and affords a point of departure for determining the angle of rotation at failure of the rigid slab portion situated outside the shear crack. Thus, the theory gives expression to the fact that the angle of rotation at failure decreases as the amount of reinforcement increases and as the diameter of the column becomes smaller.

A method of slab design with reference to punching is presented at some length. This method is brought into relation with the design of the slab as a whole with reference to failure due to moments.

Some shear tests on reinforced concrete beams without shear reinforcement

R. Taylor, *Magazine of Concrete Research* (London), V. 12, No. 36, Nov. 1960, pp. 145-154
Author's Summary

Tests were made on rectangular reinforced concrete beams without shear reinforcement to determine the effect on shear strength of manner and position of loading, amount and type of reinforcement, and strength of concrete. The results indicate that:

(1) For short shear spans, a beam loaded directly on its top surface can sustain a load greater than that causing the formation of a major inclined crack (the diagonal cracking load), but only with considerable widening of the crack.

(2) A beam loaded through transverse beams or nibs will not sustain a load higher than the diagonal cracking load, whatever the shear span.

(3) The diagonal cracking load increases slightly with increase in the amount of longitudinal reinforcement, but is apparently little affected by the bond characteristics of the bars.

(4) The diagonal cracking load increases with increase in the strength of the concrete, but not at the rate indicated by the British Standard Code of Practice CP 114:1957.

In an evaluation of the results of the tests a method is developed for estimating the diagonal cracking load of rectangular beams without shear reinforcement. From a comparison with available test data it is shown that, though the method is simple in its application, it has a fair degree of accuracy. Design recommendations based on it are given.

Surface cracking of mass concrete structures at early form removal (in Swedish)

Torben C. Hansen, *Applied Studies* No. 4, Swedish Cement and Concrete Research Institute at the Royal Institute of Technology, Stockholm, Sweden, 1960, 66 pp., 10 Kr.

A theoretical and experimental investigation was made of the problem of surface cracking of mass concrete.

On the basis of a rheological model, with a deformational behavior similar to that of concrete, the criterion of crack formation was developed. In equation form this criterion gives the relationship between rate of deformation, tensile strength of concrete, deformational properties of concrete, and time, the four primary factors in the problem of crack formation.

According to this criterion, strain at rupture of concrete varies with the stress and strain recordings before rupture. If the shrinkage or decrease in temperature occurs at a low rate, strain at rupture is much larger than if the rate of shrinkage or decrease in temperature occurs quickly. This is exemplified by inserting in the criterion of crack formation the experimentally determined rheological constants of a normal concrete and calculating the strain at rupture at different rates of temperature changes.

Only shallow and harmless cracking occurs in the surface of a mass con-

crete structure when it is exposed to drying shrinkage. The large temperature change which happens in winter when removing forms creates tensile stresses at greater depth and causes deeper and more severe cracks. As an example two cases have been treated theoretically: the influence of temperature change at form removal on a completely free mass concrete wall, and the influence on a completely restrained wall. While the surface of a free wall is severely stressed when forms are removed soon after casting, conditions are more favorable in the restrained wall. In the last case the compressive stresses induced in the wall when temperature first raises give a safety margin when temperature subsequently decreases.

The theoretical conclusions were verified in two field tests. In the first three test specimens were cast under winter conditions in northern Sweden. The specimens were 2 m high, 6 to 8 m large and respectively 1 m, 2 m, and 4 m in width. The forms were insulated by two layers of empty cement sacks, except on one section where the concrete was cast against an ordinary 22 mm wooden form without any extra insulation. A large number of thermistors were embedded at the different sections. The forms were stripped at different times and a thorough observation of the surfaces revealed an extensive crack formation on the 2 and 4 m specimens when forms were stripped at an early age. No crack formation was observed on the 1 m specimen even when forms were stripped at an early age. In the main our theoretical conclusions were confirmed by the results of this test.

In the second field test a concrete wall was built under similar conditions as in the first test but the cement content of the concrete used was somewhat lower. The observed crack formation was much smaller than in the first case.

Practical conclusions are drawn from the results of the investigation.

Development of microcracks in concrete

R. Jones, *RILEM Bulletin* (Paris), No. 9, New Series, Dec. 1960, pp. 110-114
Author's Summary

Presents the evidence which has shown that microcracks occur in concrete specimens prior to failure and considers the influence of the propagation of these cracks on the ultimate failure of the concrete.

The detection of microcracks can be made by the following methods: (1) microscopic examination of the exposed faces of the concrete specimens; (2) measurement of the strain distribution along one of the exposed faces of a specimen; and (3) measurement of the change in the transit time of an ultrasonic pulse as it passes through the concrete between two opposite faces of a specimen.

Load distribution in composite girder-slab systems

G. Hondros and J. G. Marsh, *Proceedings*, ASCE, V. 86, ST11, Nov. 1960, pp. 79-109
Authors' Summary

A series of simply supported composite girder-slab systems embracing five and ten girders interconnected by a reinforced concrete slab was tested, from which confirmation of certain assumptions made in the design of a prototype bridge was sought. Comparisons between actual and theoretical behavior are made and the effect of skew angle of the bridge and torsional stiffness of the girders is assessed. The work brought to light the fact that the neutral planes of the girders are neither coplanar nor stationary and an attempt is made to explain this phenomenon, particularly from the point of view of induced distributed longitudinal forces in the slab, which undoubtedly occur in these structural types.

Primarily, the experimental work was aimed at providing an assessment of assumptions made with regard to distribution of loads and the effects of a 30 deg skew angle on a ten-girder, 130-ft composite steel girder and con-

crete deck bridge span. Following the examination of the first model, two additional types were tested in which all properties were retained except that in the second model the torsional stiffness of the beams was reduced by 50 percent, and in the third model the span was not skewed. From each of these main models, two five-girder systems, cracked and uncracked, were obtained by cutting the former along their longitudinal centerline.

Pressure of fresh concrete on formwork (in Dutch)

A. M. N. VAN EYLE and L. BOERSMA, *Cement* (Amsterdam), V. 11, No. 3, June 1959, pp. 262-266

Reviewed by JOHN W. T. VAN ERP

During construction of a water storage tank, pressure on the formwork of the wall was measured as against the bottom of the wall formwork with a pressure membrane box and also by the deflection of the steel formwork ribs with strain gages. Pressure as a function of time was plotted.

Testing and applications of precast concrete sandwich elements (in Swedish)

AKE HOLMBERG and ERIK PLEM, *Nordisk Betong* (Stockholm), No. 4, 1960, pp. 277-304

Reviewed by MARGARET CORBIN

The article deals with two full-scale loading tests on precast concrete sandwich elements of a type described by Holmberg in *Nordisk Betong*, 1958, No. 4. An unintentional eccentric position of the second specimen in the testing machine led to a more detailed study of the effect of eccentricity. Deformations and forces have been calculated for typical cases and charts plotted for calculation of design stresses. Probable eccentricities which are opposite in direction in a sandwich element used as a load-bearing wall are discussed and a proposal for calculation of relative eccentricity and for permissible stresses is advanced. A numerical example is worked out.

Structural interaction of walls and floor slabs

SVEN SAHLIN, *Bulletin* No. 33, Division of Building Statics and Structural Engineering at the Royal Institute of Technology, Stockholm, 1959, 202 pp.

This study of the load-carrying capacity of structural frames of buildings deals with the following frame types: (1) brick masonry walls and cast-in-situ concrete floor slabs; (2) prefabricated lightweight concrete element walls and prefabricated lightweight concrete element floor slabs; and (3) monolithic reinforced concrete frames.

The angles of rotation of the ends of brick masonry walls were investigated. A theory which takes account of the effect produced by deflections on the effective cross section of the wall, and hence on the angles of rotation, was compared with theories in which the effect of deflections is disregarded.

Theoretical investigations as well as tests demonstrated that the interplay of forces in structural frames of the type under consideration and the load-carrying capacity of such frames are substantially influenced by plastification, cracking, and load-carrying capacity of the joints which connect walls and floor slabs.

It was observed that the floor slab has a marked stabilizing effect on the brick masonry immediately below the slab. Special tests were made to study this effect.

The results obtained from the full-scale tests were used to evolve approximate methods for calculating the load-carrying capacity of brick masonry walls forming part of structural frames. These methods take account of the effect produced on the load-carrying capacity by the plastification of joints.

A similar study was carried out with a view to calculating the load-carrying capacity of lightweight concrete wall elements. The angle of rotation and the load-carrying capacity of the walls were examined in this case too. Lightweight concrete wall elements can take relatively high tensile stresses, and this fact implies certain modifications in

the basic design assumptions, which were taken into account in this connection.

The main difference in behavior between these joints and those which connect brick masonry walls and cast-in-situ concrete floor slabs was that the floor slab in this case did not produce any stabilizing effect on the wall. Lightweight concrete element walls and floor slabs are connected with the aid of mortar joints. Therefore, when the eccentricity of the load was small, the load-carrying capacity of the wall was lower than that of a wall without mortar joints. On the other hand, the plastifiability of the joints between walls and floor slabs was found to be high.

The angles of rotation of the joints were considerable in both Types (1) and (2). It is shown how important it is to choose an appropriate value of the ratio of the flexural rigidities of walls and floor slabs if it is desired to prevent large cracks. It is also essential that the clearance in the joint or the plastic deformation of the joint should be taken into account in calculating the eccentricity of the load acting on the walls and the load-carrying capacity of the walls.

The last part of the thesis deals with joints of reinforced concrete frames with over-reinforced vertical members and with horizontal members whose strength is excessive relative to vertical members. In contradistinction to the joints of the Types (1) and (2), the joints of reinforced concrete frames are able to take tensile stresses.

The stress distribution in the corner region of a frame under elastic conditions was studied with photoelastic models. It was found that the critical cross section of frame corners was situated a distance from the internal corner in spite of the fact that the absolute values of the stresses were higher at the internal corner itself.

Tests were made to study the angle of rotation of a reinforced concrete frame corner (joint). The results indicate that the angle of rotation of the corner rectangle itself is about equal to the angle of rotation of a straight beam whose length is equal to the sum of the lengths of the vertical and horizontal centerlines in the corner rectangle.

Tests were carried out on two-hinged reinforced concrete frames for the purpose of studying the variation in the eccentricity of the load with the magnitude of the load applied to the frame. Just as in the tests on brick masonry walls, the area of failure in the vertical frame members was some distance from the bottom surface of the horizontal frame member, but was here in a region of the plastic hinge type. The angle of rotation of the plastic hinge was measured in the tests. The ultimate loads of the frames were 6.8 to 8.7 times as great as the corresponding permissible loads calculated by the elastic theory on the basis of the permissible stresses stipulated in the current Swedish standard specifications. These ultimate loads were 10 to 25 percent lower than those which were calculated by limit design by using the values of the maximum moments at critical cross sections obtained from the comparison tests. The fact that the load-carrying capacity was lower than that calculated by limit design seemed to be due to the circumstance that the rotation capacity of the plastic hinges had been exceeded in the tests.

General

Small size pipe for sanitary lateral sewers

Building Research Advisory Board, *NAS-NRC Publication* No. 507, National Academy of Sciences-National Research Council, Washington, D.C., Revised Edition, 1960, 48 pp., $1.50

This Building Research Advisory Board report to the Federal Housing Administration is a revision of a report originally issued in 1957. The report makes recommendations regarding the minimum size pipe to be allowed for use in street sanitary sewers in

residential areas and the conditions which should be attached to the use of minimum size pipe. Recommendations are also made for good design, construction, and maintenance practices relative to the use of small size sanitary sewers.

36 books every ready mix producer needs!

Concrete Products, V. 63: No. 6, June 1960, pp. 45-50, 53-54; No. 7, July 1960, pp. 42-45; No. 8, Aug. 1960, pp. 43-44, 47-48, 51; No. 9, Sept. 1960, pp. 39-42, 46-49; No. 10, Oct. 1960, pp. 45, 47-48, 50, 52, 54

As the ready-mixed concrete industry strives toward better quality and higher standards for its product, it becomes increasingly important for a producer to build an intelligent reference library. The editors of *Concrete Products* prepared a basic list of publications that they feel should be included on the bookshelf of every ready-mixed concrete producer.

To be included here, a publication had to provide either engineering data to help a producer manufacture a better product or it had to contain information to more fully acquaint the reader with the standards or specifications he should meet. Parts 1-3 review books and pamphlets that a producer must purchase. In Parts 4 and 5 are listed publications that may be obtained free of charge in the United States.

Papers on soils 1959 meetings

Special Technical Publication, No. 254, American Society for Testing Materials, Philadelphia, 1960, 376 pp., $9

Contains three symposiums and one group of papers dealing with various aspects of soils testing designed to help engineers evaluate soils and their construction applications. Includes numerous references and extensive graphical material.

Included are: Symposium on Time Rates of Loading in Soil Testing; Symposium on Atterberg Limits; Papers on Soils presented at the ASTM Sixty-Second Annual Meeting; and Symposium on Soils for Engineering Purposes presented at the ASTM Third Pacific Area National Meeting.

The Symposium of Time Rates of Loading deals with consolidation of clays and soils, evaluation of soils strain-rate effects and contains an extensive literature survey on dynamic and static resistance of cohesive soils for the period 1846 to 1958.

Various aspects of the well-known Atterberg Limits Tests are treated in that portion of the book. The additional papers cover aspects of soils and soils testing as soil-cement mixtures, soil stabilization, moisture-density tests, and triaxial compression tests. In all the reader will find 32 papers in this extensive and comprehensive volume.

1960 Building products register AIA

American Institute of Architects, Washington, D.C., 1960, 700 pp., $25

The American Institute of Architects has published this Building Products Register with more than 1300 product listings. The intent is to bring together and summarize in one convenient place the reference material for each product type. The product is classified according to type, category, and subcategory and is described as fully and accurate as possible from data supplied by the manufacturer. Many categories are presently contained in the register and others will be added later — structural systems, curtain walls, masonry, wood, metals, glass and plastics, roofing and siding, masonry and concrete treatments, thermal insulation, sound control, flooring and wall covering, surfacing and paneling, doors, windows, hardware, partitions and wirework, furnishing and special equipment.

Immediately following the product listings in each category, technical reference material for each product type is brought together in the "Abstracts of Standard Documents." The standards have been furnished by government agencies, testing laboratories, fire underwriters, professional and scientific societies, and producer associations issuing or recommending standards.

Concrete plant standards

Concrete Plant Manufacturers Bureau, National Ready Mixed Concrete Association, *Concrete*, V. 68, No. 6, June 1960, pp. 20-21, 34, 36-37

Describes ready-mixed concrete plant equipment with the object to standardize rated capacities, the basis for determining rated capacities and certain other features of concrete plant components. The following items of equipment are covered: batchers for aggregates, batchers for cement, batchers for water, bins or silos for aggregate, bins or silos for cement, bins or silos for the separate storage of both aggregate and cement, belt conveyors for aggregate, bucket elevators for aggregate, and bucket elevators and other types of conveyors for cement.

Films and film strips showing the use of cement and concrete

Cembureau, Malmo, Sweden, Oct. 1959, 51 pp.

A list of films and film strips covering the use of cement and concrete divided into the following categories: technology and practice, manufacture of cement, surface treatment, building and structural work, highways and airfield runways, agriculture, railways, and miscellaneous. The films are compiled by country and include Austria, Belgium, France, Germany, Great Britain, Italy, Netherlands, Norway, Sweden, Switzerland, and USSR.

Concrete engineering handbook

William S. La Londe, Jr. and Milo F. Janes, McGraw-Hill Book Co., New York, 1961, 1216 pp., $25

An extensive collection of contributions on design and construction techniques related to concrete. Divisions included are materials, ordinary concrete construction, precast and prestressed concrete, structural theory, torsion, chimneys, earthquake resistant design, building frames, estimating pavements and bases, foundations and retaining walls, bridges, shell structures, and bins and silos. Generous use of drawings adds to the usefulness of the data presented.

Concrete day 1959 (in Dutch)

Cement (Amsterdam), V. 11, No. 6, Dec. 1959, pp. 535-541

Reviewed by John W. T. Van Erp

This yearly conference in the Netherlands, held for the second time, had four speakers on projects of primary importance: (1) Prestressed bridge at Wexford, Ireland, of unusual design with extremely slender longitudinal girders, no lateral ribs, the designer claiming that the material saved on these ribs can be used in the slab itself and still lead to a more economical design. (2) The operation of the SECO office in Belgium, a cooperatively maintained office of building supervision. (3) Use of prefabricated concrete elements in large building and stadium construction by Nervi. (4) Construction of the Grande Dixence Dam in Switzerland, the highest in the world.

Elementary engineering mechanics

Eugene George Key, John Wiley & Sons, Inc., New York, 1960, 457 pp., $5.50

This book was developed to be employed as a text in technical courses by students who expect to go into technical work after taking a 2-year curriculum. It can also be used as a reference for students of more advanced courses and for home study. Users of this book require a knowledge of mathematics up to and including plane geometry and second year high school algebra plus the ability to learn and to use trigonometry.

Part I deals with the subject of statics or stationary bodies. Part II covers the subject of dynamics or moving bodies. Many examples are given in the text to illustrate the application of the theory to actual force systems. Simpler methods are always emphasized. Sometimes both conventional and simplified methods of solution are given.

The book appears to develop the subject clearly and thoroughly and should be especially helpful to those individuals with the specified background.

AUGUST 1961

NEWSLETTER

memo from the **President:**

UNIFORMITY

As THIS YEAR WE PLACE our emphasis on better concrete inspection and workmanship, it is worthwhile to recall that one of the prime ingredients of quality is uniformity, and reflect briefly on what it takes to get it. First, however, there must be a recognition of its value.

So often where deterioration from some cause makes its appearance, some parts are affected and others are not. Plainly, if all had been uniformly up to the standard of the best, which was built presumably at no greater cost than the poorest, many more years of presentable serviceability would have been obtained. Many things can cause such conspicuous variation in ultimate quality and durability. Prominent among them are variations in water content, water cement ratio, and slump due to initial variation in materials, or to later water gain and bleeding. Variation in air content alone can cause an evident lack of uniformity in durability. Variation in winter protection and in curing, from ample to insufficient, can also show up later in disappointing performance.

Thus it appears there must be acceptance of the necessity for explicit specifications and for the forthright inspection that good specifications are meaningless without. Requirements for such specifications are stated in the ACI standard "Recommended Practice for Measuring, Mixing, and Placing Concrete (614-59)," and other official standards including those of ASTM for concrete materials. Inspection practices and precautions are given in the *ACI Manual of Concrete Inspection.* When these guides are followed, quality and uniformity are sure to be reflected in long serviceability with a minimum of maintenance.

One way many a job can be supplied with uniform concrete, is from the many well equipped and well controlled ready-mixed concrete suppliers who furnish a large portion of concrete used in urban construction. Many of these concerns furnish strength specification concrete with remarkable uniformity and control. They do their own con-

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trol testing and know what they are doing. They are aware of the practical meaning of the coefficient of variation and that when it is low, a money saving reduction in the required average strength can be safely made. Discriminating buyers will realize that concrete supplied from such plants contains the additional quality of uniformity—as well as reliability—and that for these reasons it may be worth considering even when such plants are not quite the low bidder.

A word about a little extra cost must be mentioned because so often good intentions are frightened away when someone says "it will cost more." Seldom, however, is it said how much or how little more, and seldom is it proved that actually the cost is more. Usually the extra cost for a worthwhile precaution is minor and probably one of the best investments anywhere, when the returns in longer life, reduced maintenance and greater uniformity in service are considered.

It might be well to look closely when "it will cost more" interferes with a serious effort to get a first class job. Is there a full understanding of the merits of the extra cost, if in fact it exists? Has the idea of more cost been mentioned merely to divert support for essential practices? It should be remembered in this connection that many things that cost a little more are well worth the difference. As far as the few requirements on which concrete quality and uniformity depend are concerned, they are already in most specifications. Presumedly bids are made accordingly. All that is necessary is to provide effective inspection to see that these requirements are carried out.

Lewis H. Tuthill
President

University of Waterloo seeks ACI back volumes

R. Green, lecturer, Department of Civil Engineering, University of Waterloo, Ont., Canada, is anxious to locate a set of bound or unbound volumes of the ACI *Proceedings* for their Engineering Library.

He would like to hear from ACI members having a set of bound or unbound volumes which they would be willing to sell or possibly donate to this growing university. The university is prepared to handle costs of binding and transportation to the library. Please contact Mr. Green before forwarding back volumes.

"Sliding" Forms Used In Bridge Deck Reconstruction

Sliding forms and large sheets of prefabricated wire reinforcement saved time and money in the recent reconstruction of the concrete deck on Memorial Bridge carrying Route U.S. 1 from Portsmouth to Kittery, Maine. Rebuilding the deck was the single largest part of the 1960 overhaul and renovation of the 37 year old span, owned and operated jointly by Maine and New Hampshire.

The use of "sliding" plywood forms for the 5 in. concrete deck, saved time and labor on the job.

Stripping old deck

The first task to face the contractor was stripping the old deck (deeply pitted, in some places broken completely through) from the structural steel stringer and beam system of the bridge, approximately 900 ft long, excluding approaches. This was accomplished in 12 days, using paving breakers and crane, the broken material being hauled to the adjacent banks of the fast tidal-running Piscataqua River, where is is used as rip-rap.

Four lines of 4-in. tees, buried in the old slab for the south bound lane, support for street car tracks which formerly crossed on the bridge, complicated and slowed the deck removal a bit. Torches had to be used to cut the tees away from the transverse I-beams which supported the deck.

Forming new deck

When the old slab was out of the way, no time was wasted in forming for the southbound lane of the two lane bridge. The pressure of a 17,000 daily traffic count through and between the two cities dictated that at least one lane be restored to traffic as soon as possible.

Because the transverse 7 in. I-beams extend in unbroken single lengths across the five longitudinal stringers which carry the weight of the truss spans, the contractor saw an opportunity to save forming time and money. He formed the deck for one lane with ¾ in. plywood sections (stiffened by three 2 x 4's), just wide enough to ride the flanges of the 2 ft 5 in. centered I-beams, and about 15 ft long or half the bridge width. These sections were wedged into place from underneath the bridge, tight against the top flange of the I-beams.

In forming for the northbound lane, the contractor's men simply pulled the wedges under the forms for the previously cast slab, broke bond, and let the plywood sections drop down onto the lower flanges of the I-beams. The form was then slid into place for the northbound lane, one man pulling it readily from above with a light line wrapped around a nail in the form (Fig. 1).

Fig. 1 — Plywood form dropped from previously concreted lane is slid into place for second lane. Reinforcement protruding from slab will overlap reinforcement from near lane

The contractor estimates that he saved 15 days of forming time by this simple expedient, and made double use of his plywood.

With the deck forms secured, the next step was to place 4 x 6 in. creosoted wood beams and start forming the curbs, which were to be cast monolithically with the deck slab. The creosoted beams butted up alongside the outside of the curb (to which they were anchored with ¾ in. hook bolts), and were topped by the inside ends of the existing plank sidewalks of the bridge.

Wire fabric reinforcement

Next step was to place the sheets of welded wire fabric reinforcement for the deck slabs. The 3 x 6—00/00 fabric was delivered to the job in 10½ ft lengths 87 in. wide. The 3 in. spaced wires were longitudinal and placed longitudinally in the deck so that they spanned the space between the transversed beams and provided the structural reinforcement. The 6 in. spaced transverse wires mainly met temperature requirements of the slab across the bridge width.

The bottom layer of steel fabric sheets was supported away from the I-beams and plywood forms by 1-in. chairs or bolsters. The top layer of fabric was supported by small blocks 2 in. concrete brick scattered at random on the bottom layer (Fig. 2). With place, ½ in. of concrete cover (from the top of the slab) resulted as specified.

Photos courtesy Wire Reinforcement Institute

Fig. 2 — Top layer of reinforcement being placed on concrete brick scattered at random on lower layer. Formwork for curb is being constructed in background

The two widths of fabric required to make up a lane width (14 ft), were edge lapped 3 in., with the end laps between lengths of fabric being 6 in. To avoid an unwieldy pile-up of steel thickness at the laps in the middle of the lane, the laps between sheet lengths were staggered, with the bottom layer of reinforcement started with a half length, the top with a full length.

To assure continuity of reinforcement across the entire bridge deck, the south bound or first lane to be concreted, was built 6 in. short of full lane width so that the edges of the wire fabric sheets protruded out of the slab. Sheets for the northbound lane were lapped with these edged, and the concrete for that lane butted against the previously placed concrete. With the fabric sheets in place for the deck, bars to reinforce the curb were positioned and tied, and the curb formwork was completed.

Average time for placing the fabric sheets was 8 hr per 300-ft span. It's estimated that placing an equivalent weight of individual reinforcing members would have taken 2½ to 3 days.

Concreting

Concreting the decks was the next operation, with two 300-ft spans (single-lane) being completed in a day. (The bridge consists of a series of approach spans on the Maine side, two truss spans, and a lift span. The lift span has a steel grid deck, and required no reconstruction.)

The ready-mixed concrete was placed directly from the trucks, hand spread, machine vibrated, and manually struck off, using two lines of pipe tied to the fabric as screed. The pipe was later pulled out. Because the contour of the fabric sheets and the pipe screeds conformed to the support beams which

were cambered, the surface of the concrete as struck-off, had the proper crown profile.

The surface was rough finished with a wood float, a coarse texture being desirable to help bond the asphaltic concrete surface to follow. The concrete was a 6.4 bag mix, ¾ in. maximum size stone, testing to not less than 3000 psi in 28 days. An air-entraining agent was added, ¾ ounce per bag, and, due to the exposed position of the slab and winds, a retardant was employed, 2 ounces per bag.

After the concrete deck took initial set, the surface was mopped with asphalt, a layer of Fiberglas film applied and mopped, a second layer of film applied and again mopped. With this waterproofing in place, 1 in. of asphaltic concrete was machine placed, and the lane was then opened to traffic.

The same sequence of operations was followed for construction of the northbound lane, with one-way controlled traffic being maintained over the southbound lane except for the day or days when concrete was being delivered to the job. When both lanes were completed, a final 1 in. wearing course of asphaltic concrete was placed over the entire bridge width.

Work started September 6th and the bridge deck, curbings, and installation of drainage scuppers on the approach span were completed by November 8th.

Credits

Harond E. Langley, bridge engineer, New Hampshire Highway Department, was coordinator for the two states. General contractor was Monroe-Langstroth, Inc., Norwood, Mass.

Burns to manage Sika Denver office

Donald F. Burns will manage the new district office of the Sika Chemical Corp., which will be referred to as the Rocky Mountain District office, located in Denver.

Ferguson and Thurlimann 1961 ASCE awardees

ACI past president Phil M. Ferguson, professor of civil engineering, University of Texas, Austin, was among five winners of American Society of Civil Engineers Research Prizes for 1961. Professor Ferguson was cited "in recognition of his outstanding contribution to knowledge of the behavior of concrete structures."

Professor Ferguson has been in the Department of Civil Engineering at the University of Texas since 1928 and was department chairman from 1943 to 1957. His researches have contributed materially to knowledge and understanding of ultimate strength design concepts. He has authored many research papers and a recent textbook, *Reinforced Concrete Fundamentals*, has become a technical bestseller.

Long active in ACI affairs, Professor Ferguson is currently a member of ACI Committee 115, Research; Committee 208, Bond Stress; and Committee 318, Standard Building Code. He is ACI representative on the American Welding Society Committee on Welding Concrete Reinforcing Steel.

Bruno Thurlimann, professor of structural engineering, Swiss Federal Institute of Technology, Zurich, Switzerland, was cited for "his outstanding contribution to the development of plastic design concepts." Dr. Thurlimann did graduate work at Lehigh University, Bethlehem, Pa., receiving his doctorate in civil engineering in 1951. In 1952 he was research associate with the Graduate Division of Applied Mathematics at Brown University, Providence, R. I. and from 1953-58 was on the civil engineering staff of Lehigh University. He has been at the Swiss Federal Institute since 1959.

Dr. Thurlimann is currently a member of ACI-ASCE Committee 333, Design and Construction of Composite Structures, and ACI Committee 334, Concrete Shell Structures.

Roll appointed chairman of ACI Committee 209

Frederic Roll, professor, The Towne School of Civil Engineering, University of Pennsylvania, Philadelphia, was appointed chairman of ACI Committee 209, Creep and Volume Changes in Concrete, by the Board of Directors at the 1961 annual meeting in St. Louis.

Professor Roll is a member of ACI-ASCE Committee 326, Shear and Diagonal Tension.

A graduate of Columbia University, he previously taught at the College of the City of New York and Columbia University. He has also worked in private engineering firms as a stress analyst, and highway and highway bridge designer.

JOURNAL goes multilingual

Beginning with the July, 1961, issue of the new Proceedings volume, the JOURNAL is carrying Spanish, French, and German translations of paper titles and synopses. The synopses are direct translations of the English synopses, which have long been an important JOURNAL feature, and appear together following the paper. Papers continue to be published only in English.

The new step was undertaken to make the JOURNAL of greater value to ACI members overseas.

A large number of ACI members, about one in four, live outside the United States. The additional synopses now make the copies of the JOURNAL already going overseas of greater value to more concrete technologists. Now persons overseas will be able to know as soon as each JOURNAL arrives whether it contains information pertinent to his current undertakings. Spanish-, French-, and German-language publications will also find the synopses of value in keeping their readers informed, through their review and abstract sections, of works published in the JOURNAL.

ACI technical committee appointments

Listed below are committee members who have recently accepted appointments to ACI technical committees. Included are *new* appointments only.

Committee 115, Research
Ralph W. Kluge
University of Florida
Gainesville, Fla.

Committee 213, Properties of Lightweight Aggregate and Lightweight Aggregate Concrete
John K. Selden
Concrete and Autoclave Products
Toledo, Ohio

Committee 341, Reinforced Concrete Columns—Joint ACI-ASCE
Albert C. Bianchini
University of Illinois
Urbana, Ill.

Bengt Broms
Cornell University
Ithaca, N.Y.

Roger Diaz de Cossio
Instituto de Ingenieria
Ciudad-Universitaria
Mexico, D.F., Mexico

John J. Kozak
California Division of Highways
Sacramento, Calif.

J. G. MacGregor
University of Alberta
Edmonton, Alta., Canada

Ignacio Martin
SACMAG of Puerto Rico, Inc.
Santurce, Puerto Rico

A. S. Neiman
Consulting Engineer
Beverly Hills, Calif.

Howard Simpson
Simpson, Gumpertz & Heger, Inc.
Cambridge, Mass.

Mete A. Sozen
University of Illinois
Urbana, Ill.

C. A. Willson
American Iron and Steel Institute
New York, N.Y.

Bethlehem also furnished the fabricated reinforcing bars used on this section of freeway.

Bethlehem Stress-Relieved Strand in 24,000 lineal feet of hollow prestressed-concrete piling for Seattle Freeway

High above Lake Union, in the Capitol Hill area of Seattle, Washington, a 2,700-ft section of the new Tacoma-Seattle-Everett Freeway is being rushed to completion. It consists of three pile-supported, raised roadways, side by side, each with 4 lanes. The center viaduct is reversible to handle peak traffic loads.

Production line system was set up by contractor. Several piles were made at one time, end-to-end. After curing cycle, they were separated by burning the strands between piles.

Morrison-Knudsen Co., Inc. and Rumsey Co., handling the contract as a joint venture, used over 900 tons of 7/16-in. diameter Bethlehem stress-relieved strand in the fabrication of 973 prestressed concrete girders and 405 hollow prestressed piles. A typical bent is shown above. The 5 cylindrical piles support 13 I-section girders, which are placed across the top of a cast-in-place cap. A concrete slab is then poured to complete the deck.

Leading prestressers are using Bethlehem stress-relieved strand because they like the results. They find that Bethlehem strand is flexible and easy to handle . . . and that they get the same dependable mechanical properties in every reel. Your nearest Bethlehem Sales office has full details.

BETHLEHEM STEEL

BETHLEHEM STEEL COMPANY, BETHLEHEM, PA.

Export Sales: Bethlehem Steel Export Corporation

Penn State schedules engineering seminar

The Pennsylvania State University, University Park, will conduct an engineering seminar on structural aspects of architectural engineering, September 10-15.

Four basic areas of study will be covered during the seminar: analysis and design of structural systems; fire protection aspects of structural systems; modular coordination; and new developments.

Subjects to be discussed include a review of the development of basic concepts of ultimate strength design for concrete and plastic design for steel; design of elements and structural systems based on ultimate load theory; review of latest developments relating to the influence of fire protection on the design of structural systems; review of latest developments relating to the influence of modular coordination on the design of structural systems; and review of new developments relating to structural aspects of architectural engineering, including structural plastics, laminates, new materials, and atomic shelter.

Seminar cochairmen are Gifford H. Albright, associate professor of architectural engineering, and Melvin W. Isenberg, associate professor of architectural engineering.

Futher information may be obtained from Architectural Engineering, 133 Hammond Bufiding, The Pennsylvania State University, University Park, Pa.

WANTED! Photos or slides of faulty concrete construction

ACI Committee 611, Inspection of Concrete, is engaged in collecting a series of slides illustrating the need for adequate concrete inspection.

Do you have a clear photograph or slide showing faulty concrete construction which you could contribute to the library of slides being assembled? The projects need not be identified further than an appropriate description of what caused the lack of acceptability.

Address: S. D. Burks, ACI Committee 611, c/o W. R. Grace and Co., Dewey and Almy Chemical Division, 2140 Davis St., San Leandro, Calif.

Five ACI members awarded CSI honors

Five ACI members were among those paid special honor at the Construction Specifications Institute fifth annual convention in New York, May 22-24.

Harry C. Plummer, Structural Clay Products Institute, Washington, D. C.; Rolf T. Retz, California State Division of Architecture, Sacramento; and J. Stewart Stein, architect, Franklin Park, Ill., were made Fellows in recognition of their achievements and service to the institute.

President's plaques were presented to J. Donald McFarlan, architect, Atlanta, Ga., for his work in handling local construction problems, and to Terrell R. Harper, architect, Harper and Kemp, Dallas, for his initiative and effort as a national director of CSI.

Best elected president of Consolidated Rock Products

Quentin W. Best was recently elected president of Consolidated Rock Products Co., Los Angeles. He succeeds Robert Mitchell, who is now chairman of the board.

Mr. Best started his career as a salesman when Consolidated Rock was first organized in 1929. During the years he has served as assistant sales manager, general sales manager, vice-president and director, and executive vice-president.

Mr. Best has been a director and member of the executive committee of the National Ready Mixed Concrete Association since 1958 and at the annual convention held last January he was elected treasurer.

Prestressed Bull Ring Erected in Only 90 Days

THE PLAZA MONUMENTAL DE TIJUANA, Mexico, was erected in the incredibly short time of only 90 days. This remarkable engineering feat is accented by the fact that traditionally constructed bull rings take anywhere from 2 to 10 years to build. Much of the credit for the speedy construction was due to the fact that this 23,000-seat Plaza utilized precast prestressed concrete.

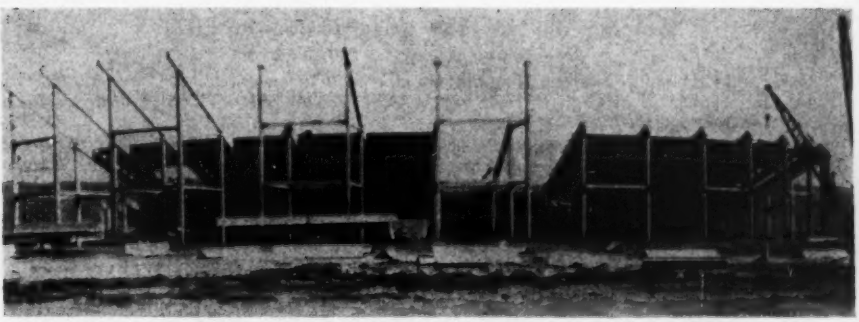
The project was launched on Feb. 6, 1960; on February 25, the contract was signed for the entire construction job, including complete design, fabrication, and erection of the Plaza with a guaranteed opening date of June 26.

Midway through the preliminary negogiations, the prestressed concrete portion of the bull ring was doubled in size. The seating capacity was increased from the original 18,000 figure to 23,000, box seats were added, and with other extras the Plaza was enlarged 50 percent over initial plans.

Because of the vast number and variety of prestressed members required, and the shortness of time, it was necessary to double the personnel at the supplier source, San Diego Prestressed Concrete Co., San Diego, Calif. A special transportation division was created by the company to supervise and handle the daily stream of beams, columns, seats, and other concrete members—all prestressed and precast in San Diego, keyed, and ready for assembly and erection on the stadium site 30 miles away in Mexico.

During the first weeks, while San Diego Prestressed Concrete Co. was originating the preliminary engineering and organizing yard equipment, Raymundo Muzquiz, the general contractor and head of Grava Arena y Concretos Mexicanos, a Mexican construction firm, was operating a 7-day-a-week, 24-hr-a-day crash program preparing the site. He performed the tremendous task of excavating and building the foundation and an underground utility system in 5 weeks.

The first shipment of concrete elements arrived on April 13, and on June 26 the prestressed concrete bull ring was ready for its inaugural program.



Precast prestressed concrete bull ring under construction

The Plaza's arena, or ring, is 132 ft in diameter set at an elevation of 20 ft below ground level with 40 tiers of seats, plus walkways and aisles. The exterior diameter of the huge circular stadium is 354 ft, with the highest seats 55 ft above ground level, 75 ft above the fighting ring.

The all-prestressed concrete stadium required 132 major beams, 132 columns, 154 struts, and 1980 precast seat sections totaling over 36,000 lineal ft. In addition, the job called for several hundred additional special sections of stairways, railings, and circumferential tie members.

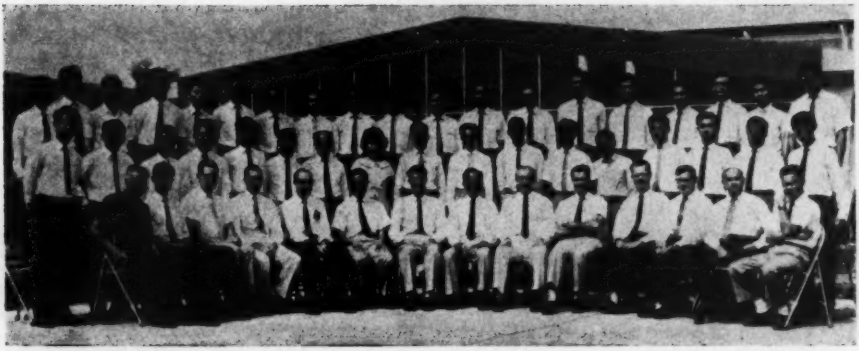
This Mexican plaza de toros is owned by Major J. Salvador Hurtado. The principal engineering architect was Jaime A. Sandoval, Mexico City; consulting engineer for the structural design was Dorland-Ferver Associates, San Diego, Calif.

Automation Hits the Construction Industry

Automation is felt in the construction industry as it affects increased production, time saving, and reduction of costs. These factors will be felt in the forthcoming apartment and high rise office buildings upsurge.

As part of its labor and time saving equipment, the A. J. Etkin Construction Co., Oak Park, Mich., is using an imported Danish crane. This new equipment makes the conventional crane of limited use on projects of this nature. Flagmen are eliminated; a safer job is assured by virtue of the operator being positioned at the working level rather than at street level. Material can be positioned exactly where needed rather than rehandling on the deck, as the crane has a 360 deg working radius. Noise fatigue is no longer a factor as the crane is electrically operated.

The first application of this equipment in the Midwest is the Nelson Tower Apartments, Jackson, Mich. This project was designed by King & Lewis, Inc., AIA, Detroit. Albert Nelson is the owner and represented by Ferdinand Otto on the job site. Field superintendent for A. J. Etkin Construction Co. is Anthony Boscarino.



Students and faculty of the SEATO Graduate School of Engineering. ACI members among the faculty are (seated): R. Kanjanavanit, Chulalongkorn University, Thailand (second from left); Elihu Geer, University of Detroit (third from left); and C. Muktabhant, Chulalongkorn University (sixth from left)

Engineering in Thailand —SEATO Graduate School

In September, 1959, on the campus of Chulalongkorn University, Bangkok, an interesting though unheralded project of international cooperation got underway—the SEATO Graduate School of Engineering. The project came about through efforts of the SEATO Ministers in 1958 and provides advanced engineering training in the hydraulics; highway; structural; and beginning in 1961-62, sanitary fields.

The school project is a cooperative effort of Australia, France, New Zealand, Pakistan, Philippines, Thailand, United Kingdom, United States, and SEATO, who provide financial support, teaching and administrative personnel, or both. Degrees are issued through Chulalongkorn University which must approve the study programs.

The 2-year program leads to the Master of Engineering degree and is open to eligible students from any of the South East Asian countries. Among the facilities provided are hydraulic and structural laboratories described as second to none in South East Asia. A highway laboratory and a library are in the process of development.

The first graduating class in March, 1961, had eight graduates in the hydraulics program. Of the eight, two have received scholarships to continue their studies toward doctorates in the United States and one has received a French Government scholarship for further study.

Most of the faculty of the SEATO Graduate School are on leave from teaching assignments in their sponsoring countries and stay for 2 years. Providing an adequate number of qualified teachers is one of the school's most pressing problems. ACI member Elihu Geer, on leave as professor and head of the Civil Engineering Department, University of Detroit, has been serving on the faculty since June, 1960. Professor Geer provided the above information.

The United States' contribution to the SEATO Graduate School includes nine staff members, major budget support, exclusive support of extension on special training program, and technical periodicals.

1961 REGIONAL MEETING
NOVEMBER 1-3
BIRMINGHAM, ALABAMA

Brother Leo retires after 31 years of service

Brother Amandus Leo for 31 years dean of Manhattan College's school of engineering retired at the end of the spring term. He has been succeeded by Robert T. Weil, formerly head of the electrical engineering department.

Although he is retiring as dean, Brother Leo has just been re-elected president of the Association of Engineering Colleges of New York State. He will continue at Manhattan College as a faculty member in the department of mathematics.

He is a member of the Mayor's Advisory Planning Board for the Bronx and the Mayor's Committee on Professional, Technical, and Managerial Manpower in New York City. He is also chairman of the engineering scholarship committee of the Now York State Society of Professional Engineers and a member of the committee on teaching aids of the American Road Builders Association.

The new dean, Mr. Weil, has been a faculty member at Manhattan for 29 years. He has been head of the electrical engineering department since 1945.

Hansen retires as Universal Atlas research director

Waldemar C. Hansen has retired as director of the Research Laboratories of Universal Atlas Cement Division, United States Steel Corp., Gary, Ind., after 16 years with the company.

Dr. Hansen will do consulting work and continue his committee work with ACI and other professional groups. He is currently a member of ACI Committee 115, Research; Committee 212, Admixtures; Committee 623, Cellular Concretes; and Committee 716, High Pressure Steam Curing.

Dr. Hansen is chairman of the American Academy of Sciences-National Research Council Advisory Panel to the Building Research Division of the National Bureau of Standards. He is also chairman of the Advisory Committee on the Long-Time Study of the Performance of Portland Cement in Concrete of the Portland Cement Association.

Stelson named ALCOA professor of engineering

J. C. Warner, president, Carnegie Institute of Technology, Pittsburg, has announced the appointment of Thomas E. Stelson as ALCOA professor of engineering. The ALCOA professorship, established in 1953, is sponsored by the Aluminum Co. of America.

Professor Stelson received his BS, MS, and DSc degrees from Carnegie Tech and has been a member of the Carnegie faculty since 1952. From 1957-59 he was acting head of the department of civil engineering and since 1959 has been head of the department.

Dr. Stelson has written articles in leading professional journals as well as the ACI JOURNAL.

He recently returned from Pakistan where he served as a consultant on problems of engineering education to the Ministry of Education. One result of his trip was the formation of a Pakistan Society for Engineering Education, an organization of engineering educators from throughout the country. Dr. Stelson was elected a charter member of the group.

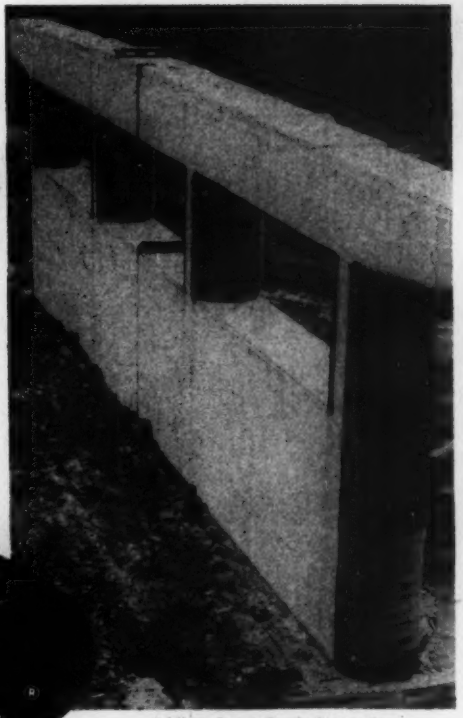
Active in ACI committee work, Professor Stelson is chairman of Committee 215, Fatigue of Concrete; and a member of Committee 115, Research. He holds membership in a number of professional societies.

Weisz appointed to WRI staff

Charles P. Weisz has been appointed structural engineer for the Wire Reinforcement Institute.

The newly named WRI engineer is a University of Colorado graduate. For the past 9 years he has been associated with Hazelet & Erdel, Cincinnati, consulting engineers.

VERSATILE FIBRE FORMS used for obround and full-round concrete columns



Save time, labor, with SONOCO FIBRE FORMS

Addison Street Grade Separation
Northwest Expressway, Chicago, Ill.
Contractor: W. E. O'Neil Construction Co.
Architect: Illinois Highway Dept.
Engineer: Alfred Benesch & Associates

On-the-job adaptability makes SONOTUBE Fibre Forms ideal for full-round, half-round, and quarter-round concrete columns, as well as obround columns and pilasters. In addition, these versatile forms are easily sawed to fit wall or beam forms, punched for tie-in rods or anchor bolts, and cut for utility outlets.

Because they are lightweight, easy to handle, and quickly placed, braced, poured, and stripped, low-cost SONOTUBE Fibre Forms provide the fastest, most economical forming method for round columns of concrete.

Choose the type that fits your requirements: "A" Coated (for exposed columns); "W" Coated (for unexposed or unfinished columns); or Seamless (for finished columns).

Sonoco SONOTUBE Fibre Forms are available 2" to 48" I.D., in specified lengths or standard 18' shipping lengths.

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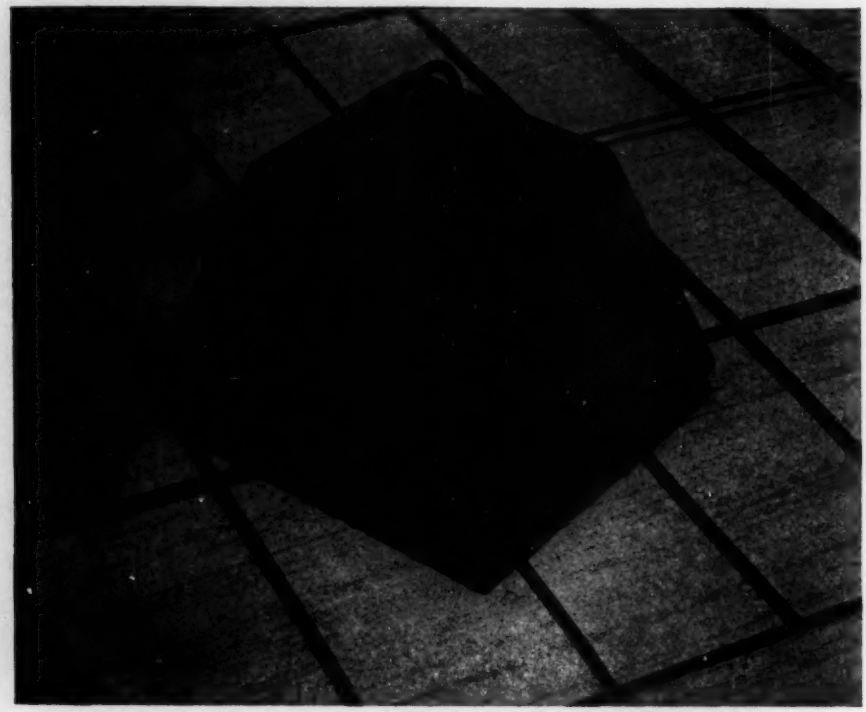

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Barinowski of Vulcan resigns from v-p post

Clarence A. Barinowski, commercial vice-president of Vulcan Materials Co., Birmingham, Ala., recently retired from this post after 44 years with Vulcan. He will continue to serve the firm as a consultant, working primarily with its Alabama and Georgia aggregates division on special projects. He will also continue as a director of the corporation.

It has been conservatively estimated that during his long years of service "The Baron" sold more than 150 million tons of local slag. His work in marketing blast furnace slag was of a true pioneering nature.

Mr Barinowski is general chairman of the ACI 14th Regional Meeting to be held at the Dinkler-Tutwiler Hotel, Birmingham, November 1-3.

He is currently a director of the National Crushed Stone Association, the National Sand and Gravel Association, and the National Slag Association. He served the latter organization for three terms as president.

Seattle world's fair symbol begun

The symbol of the first United States world's fair in more than 20 years, the 600 ft steel and glass space needle, was begun recently with an official ceremony on the site of the 1962 Century 21 Exposition. The 1962 ACI regional meeting takes place in Seattle during the closing weeks of the exposition.

State, city, and Century 21 Exposition officials took part as 2800 cu yd of concrete were placed for the 35-ft deep foundations anchoring the base of the space needle. At the dedication ceremony local citizens submitted products of today which, in their estimate, will be considered quaint relics of the past by space-age citizens of the Twenty-First Century.

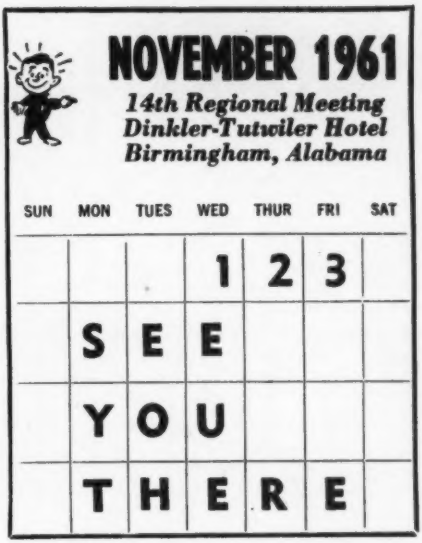
Designed by John Graham and Co., architects and engineers of New York and Seattle, the structure will be crowned by a revolving restaurant and an observation deck topped by a continuously lit beacon. The structure will be a central figure in Seattle's future civic center when Century 21 closes.

Folsom appointed chairman of BRI advisory committee

Richard G. Folsom, president of Rensselaer Polytechnic Institute, Troy, N.Y., has been appointed chairman of the Special Advisory Committee of the Building Research Advisory Board for the Study of Building Research in the National Bureau of Standards.

Dr. Folsom, an eminent administrator, and engineer, was from 1953 to 1958 professor at the University of Michigan and director there of the Engineering Research Institute. He is active in a number of professional societies.

The Special Advisory Committee has been established to review NBS functions and activities relating to building research, and, on the basis of such review and consideration of the needs and opportunities for building research in the United States, to recommend a mission or relationship which NBS



could most appropriately assume in this area.

A. Allan Bates, vice-president, Portland Cement Association, Chicago, is among 8 distinguished individuals appointed to membership on this Special Advisory Committee. Dr. Bates is currently serving on the ACI Board of Direction.

Columbia Cement to build distribution center in W. Va.

Columbia Cement Corp. will build a cement distribution point at Nitro, W.Va., along the New York Central right of way. Negotiations are underway for the purchase of a 5-acre tract for the bulk cement station. Columbia Cement, with plant at Zanesville, Ohio, is a wholly-owned subsidiary of Pittsburgh Plate Glass Co.

The new operation will consist of 3 precast concrete stave silos, 20-ft in diameter by 42 ft 6 in. high, having a total capacity of 10,000 bbl of cement. These silos will be mounted on reinforced concrete superstructure. Adjacent to the silos they will construct a concrete block scale office housing the weighmaster and truck dispatcher, as well as a remote control panel for the operation of all equipment.

A. G. Higginbotham Co., Charleston, W.Va., has been selected as contractor for the construction work. Construction will begin soon and the unit will be in operation by fall.

LOOKING AHEAD

Aug. 11-25, 1961—Prestressed Concrete Conference, Cement and Concrete Association of Australia, Sydney, N. S. W.

Aug. 30-Sept. 2, 1961 — IASS-RILEM Shell Colloquim, Delft, the Netherlands

Sept. 4-6, 1961 — IASS Shell Colloquim, Brussels, Belgium

Sept. 25-28, 1961—Industrial Building Exposition and Congress, New York Coliseum, New York

Oct. 4-5, 1961—Annual Meeting, National Slag Association, Pocono Manor Inn, Pocono Manor, Pa.

Oct. 7-10, 1961—Western Building Industries Exposition, Great Western Exhibit Center, Los Angeles

Oct. 16-20, 1961 — Annual Convention, American Society of Civil Engineers, Hotel Statler, New York

Oct. 15-19, 1961—7th Annual Meeting, Prestressed Concrete Institute, Brown Palace and Cosmopolitan Hotels, Denver, Colo.

Oct. 16-17, 1961—Annual Meeting, The Carolinas Ready Mixed Concrete Association, Inc., Jack Tar Poinsett Hotel, Greenville, S. C.

Nov. 1-3, 1961 — 14th Regional Meeting, American Concrete Institute, Dinkler-Tutwiler Hotel, Birmingham, Ala.

Nov. 28-30, 1961—Fall Conferences, Building Research Institute, Mayflower Hotel, Washington, D.C.

Dec. 5, 1961—First Quality Concrete Conference, University of South Carolina, Columbia, S. C.

Dec. 5-7, 1961—Fall Conferences, Building Research Institute, Shoreham Hotel, Washington, D.C.

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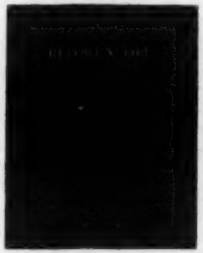

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ASTM 64th annual meeting

The 64th annual meeting of the American Society for Testing Materials was held at Chalfonte-Haddon Hall, Atlantic City, N.J., June 25-30. The 5-day meeting featured an action-packed program of luncheons, lectures, panel discussions, and some 125 papers presented at 40 technical sessions. Fifty technical committees and their subgroups met during the annual meeting.

Sessions geared to the interests of the construction industry featured a symposium on soil dynamics and sessions on soils, concrete, and road and paving materials. Nine papers were presented at the sessions on concrete.

N. B. Mitchell, Jr., Cornell University, Ithaca, evaluated the use of the indirect tension test for concrete and other brittle materials. He compared present and proposed tension testing procedures theoretically and experimentally with the indirect tension test. K. E. Palmer and I. L. Lynn, Ideal Cement Co., Ft. Collins, Colo., reported on statistical correlations of various flexural and compressive strengths of laboratory-prepared concretes and mortars using 36 Type I, II, and III cements from different parts of the United States.

J. R. Keeton, U.S. Naval Engineering Laboratory, Port Hueneme, Calif., discussed strain distribution in compressively loaded concrete specimens, determined with mechanical strain gages, resistance strain gages, and with cantilever deflection-type devices employing resistance strain gages. His discussion shed considerable light on the function of aggregate and mortar in distributing internal strains in the concrete.

S. Spinner and W. E. Tefft, National Bureau of Standards, Washington, D.C., presented a paper on methods for determining mechanical resonance frequencies and for calculating elastic moduli from these frequencies. The first part of their paper described the latest techniques and the second part described methods for computing the appropriate elastic moduli from these resonance frequencies, for specimens of the shapes most frequently used experimentally, i.e., cylindrical rods and rectangular bars.

C. M. Wakeman, Port of Los Angeles, presented a paper on application of epoxy healants for repairing structural cracks in concrete members. He described preliminary testing of the cracked concrete and field and laboratory tests leading to the selection of epoxy resins.

R. D. Vincent, Manuel Mateos, and D. T. Davidson, Iowa State University, Ames, discussed variation on pozzolanic behavior of fly ashes. The paper was based on an investigation to determine the relative effectiveness as pozzolans of different fly ashes economically available in Iowa, and to try to correlate strength of lime-fly ash mixtures with various fly ash properties by isolating groups of fly ashes having several similar properties. W. E. Grieb, Bureau of Public Roads, Washington, D.C., reported on an investigation made to determine the effect of the use of fly ash as a replacement for part of the portland blast-furnace slag cement on the strength and durability of concrete.

D. L. Bloem, R. D. Gaynor, and J. R. Wilson, National Ready Mixed Concrete Association and National Sand & Gravel Association, Washington, D.C., spoke on testing the uniformity of large batches of concrete. Modern concepts in control of concrete was the subject of a paper presented by J. J. Waddell, Soil Testing Services, Inc., Chicago.

Clair elected ASTM President

Miles N. Clair, president, The Thompson & Lichtner Co., Inc., Brookline, Mass., was installed as president of the American Society for Testing Materials during the 64th annual meeting of the society in Atlantic City, N.J., June 25-30.

Dr. Clair taught civil engineering at Drexel Institute from 1923 to 1925 and then joined The Thompson & Lichtner Co. as engineer in charge of testing and inspection. He became vice-president in charge of engineering in 1928 and held the position of first vice-president and treasurer from 1930 until 1950 when he became president. He has been instrumental in the development of lightweight concrete roof slabs and the use of concrete made with fine and coarse cinders for structural purposes.

Long active in ACI, Dr. Clair is a past-director, has served on executive and technical committees and contributed several papers for JOURNAL publication. He now represents ACI on ASA Sectional Committee A1, Specifications and Methods of Test for Hydraulic Cements. He is past-president of the New England Section, ASCE and past-president of the Boston Society of Civil Engineers. He is a recipient of the Clemens Herschel Prize Award and also the author of numerous technical papers and sections of engineering texts and handbooks. Dr. Clair also participates in many civic organizations.

ASEE membership reaches 10,000

The American Society for Engineering Education has for the first time in its history achieved a membership of 10,000.

ASEE was founded in 1893. Its membership has steadily grown from its founding size of 70 and today ASEE members are from schools, colleges, universities, research laboratories, industry, and government in the United States and at least 46 foreign countries.

ASEE is the only professional society established solely to study and improve education in its field. It has served as a pattern for engineering education societies in other parts of the world.

The society's 1961 annual meeting was held June 26-30 at the University of Kentucky, Lexington, with some 2000 members participating in the 5-day program.

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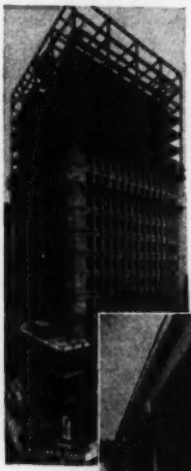

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Russell elected president of Hawaiian Cement

Walter C. Russell, president of American Cement Corp., has been named president and chief executive officer of Hawaiian Cement Corp., Honolulu, Hawaii, and Robert D. Fisher has been elected chairman of the board.

Mr. Russell will continue as president of American Cement Corp., dividing his time between Detroit and Honolulu. Before becoming president of American Cement in 1959, Mr. Russell was for 25 years president of Peerless Cement Co., Detroit, one of the companies that merged to make up American Cement Corp.

Howard R. Starke continues as executive vice-president and general manager of Hawaiian Cement Corp.

Sun-Nien joins Dewey and Almy

Sun-Nien Yu has joined Dewey and Almy Chemical Division, W. R. Grace & Co., Cambridge, as a chemist in the Construction Chemicals and Rock Products Chemicals Research Laboratory.

He was formerly with Yung Zeng Industria E. Comercio Ltda., Sao Paulo, Brazil. He received his degrees in chemical engineering from Loughborough College of Technology, England, in 1958, and Purdue University, Lafayette, Ind., in 1961.

Watson appointed Albany district sales manager

Robert G. Watson has been appointed sales manager of the Albany, N.Y., district Universal Atlas Cement Division of U. S. Steel, New York. He succeeds Harry E. Bergold, who is retiring after 20 years of service.

Mr. Watson joined Universal Atlas in 1946 as a technical service engineer in the Chicago office. He was later promoted to technical service manager, New York, a position he held until his present appointment.

He is a member of ACI, Concrete Specifications Institute, Concrete Industry Board of New York City, and Engineers Club.

Dundee Cement to construct new terminal in Michigan

Dundee Cement Co., Dundee, Mich., has purchased property north of Pontiac, Mich. for a new distribution and service center.

The new station will be located and designed to supply the increased demand for cement in the fast-growing Detroit-Pontiac-Flint industrial corridor. The new facility will substantially increase the firm's $33 million investment in the Great Lakes area.

The 2-year old cement plant currently operates distribution and service centers in Cincinnati, Cleveland, Chicago, and Des Plaines, Ill.

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De Curtoni promoted by Permanente Cement

Charles de Curtoni has been appointed general credit manager of Permanente Cement Co., Oakland, Calif. He suceeds Charles M. Chapman, who has retired after more than 16 years with the company.

Formerly division credit manager in Southern California, Permanente's new general credit manager joined the Kaiser organization in 1942.

Roberts named WRI president

D. W. Roberts, manager of sales, Construction Products, Kansas City, Mo., plant of Sheffield Division, Armco Steel Corp., has been named president of the Wire Reinforcement Institute.

The association of steel welded wire fabric manufacturers elected him at its recent annual meeting in Washington, D. C. Frank B. Brown, managing director, with offices in Washington, D. C., administers WRI's year-round promotional, research, and technical service activities.

Industrial Building Exposition to be held in New York Coliseum

Nineteen conference sessions, covering virtually every major problem of new construction and modernization, will feature the second Industrial Building Exposition and Congress scheduled for the New York Coliseum, September 25-28.

Two floors of exhibits will complement the sessions. Eight major categories will be covered in the exhibits: structural materials and components; mechanical components and equipment; electrical components, materials and systems; heating and air handling; employee services; plant sites and serviees; financing; and auxiliary services.

"Short" Proceedings volume begins

The July JOURNAL was the first issue of the new Proceedings volume as July issues have been since 1956 when the Institute began publishing 12 JOURNAL issues per year. As a result of action by the Board of Direction in October 1960, however, the July issue represents the last midyear beginning of a volume year. Proceedings V. 58 will be only 6 months long with Proceedings V. 59 beginning in January 1962. Thereafter the Institute Proceedings, represented by the JOURNAL, will be published 12 times per year January through December—with the volume being completed by the following March Part 2 third-quarter discussion and the June Part 2 containing the fourth-quarter discussion, title page, contents, indexes, and errata.

JOURNAL volume number deleted

Another change to be noted beginning with this Proceedings V. 58 is the deletion of the JOURNAL volume number. The JOURNAL will be known only by its Proceedings volume number from now on. The deletion of the dual volume numbers should be welcomed by researchers and librarians who for the past 32 years have found it difficult deciding which number to use. The use of the Proceedings volume numbers preserves the continuity of 58 years of publishing by the Institute.

Discussion unchanged

Discussion will still appear quarterly in March, June, September, and December.

T. J. Cavanagh

T. J. Cavanagh, formerly director of the Cement and Concrete Association of Australia, died on April 14.

Mr. Cavanagh was born in Brisbane in 1902. His long association with the Australian cement industry began in 1936 with his appointment to the highway staff. The following year he was named managing editor of *Construction Review*.

Mr. Cavanagh was appointed chief executive officer of the Australian Cement Manufacturers' Association in 1942 and subsequently manager in 1946 and director in 1950 of its successor, the Cement and Concrete Association of Australia. He held this position until his retirement due to ill-health in September, 1959.

Actively associated with the Master Builders' Federation of Australia, he served as honorary secretary from 1935 to 1957. He held executive posts with the Building Industry Congress of New South Wales, and was a member of the National Council of Building Industry Congresses. Active for many years with the Builders Exchange of New South Wales, he was president of the group in 1936. He was a board member of the Standards Association of Australia from 1943 to 1957 and also chairman of the SAA Cement Sectional Committee for many years.

In 1944 he was appointed by the Ministry of Post-War Reconstruction to the Technical Bodies Advisory Committee on Housing Standards, and was also (1944-45) a member of the Building Advisory Co-ordination Committee of New South Wales, appointed by the Department of War Organization of Industry. He was also a member of the Commonwealth Department of Works Building Research and Development Advisory Committee, and served as chairman for some years.

Mr. Cavanagh was awarded the Coronation Medal in recognition of his long-standing service to building in Australia.

Honor Roll

Point System

1 point for Student; 2 points for Junior; 3 points for Individual; 4 points for Corporation; and 5 points for Contributing.

Jan. 1-June 30, 1961

U. Ersoy continues as top man on the Honor Roll with 19 points to his credit.

New ACI members stem from various sources, however, the greatest number are prompted to membership by ACI members. Thousands of persons working in the concrete industry throughout the world need ACI services and ACI needs the best of their thinking. Tell your colleagues about ACI activities! Let's keep the membership trend spiraling.

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